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No. 875.

THE FOUNDATIONS OF THE NEW CROTON DAM.

By Charles S. Gowen, M. Am. Soc. C. E.

Presented February 21st, 1900.

WITH DISCUSSION.

In 1883 the Legislature of the State of New York passed an Act (Chapter 490, Laws of 1883) creating the Aqueduct Commissioners of the City of New York.

The purpose of this Act was the immediate increase of the water supply of the city which, under the conditions then prevailing, had for some time been inadequate and inefficient. To this end it was planned to begin the construction of a new aqueduct and a large dam on the Croton River, the latter near to and above the site of Quaker Bridge, at a point about 4 miles below the old Croton Dam which had been in use since 1839. This new dam, it was reckoned, would increase the available storage by about 32 000 000 000 galls., and, if construction were begun immediately, could be put to practical use, in connection with the New Aqueduct, not long after the completion of the latter, the work of which was planned to continue at the same time.

The Aqueduct Commissioners began the construction of the New Aqueduct in the fall of 1884, but found a strong opposition, on the part of a few influential citizens, to the project of the dam. This opposi-

tion resulted in an indefinite delay in action on the part of the Commissioners, so far as the large dam was concerned, but they ordered the construction of a smaller dam and reservoir near the head waters of the East Branch of the Croton, at the Village of Sodom, early in 1888. This action reversed the original plans for the enlargement of the water supply, which were, to build the large dam and basin at first and with as much speed as practicable, and later to complete the conservation of all the storage capacity of the Croton Valley by building the smaller dams and reservoirs, of which the dam at Sodom was one.

In July, 1888, a new Board of Aqueduct Commissioners came into power. They found a steadily increasing demand for more water, and came to the conclusion that it was best to continue the policy of building the smaller dams and reservoirs already inaugurated by their predecessors, as, owing to the time which had lapsed (about 4 years), without action relative to the proposed large dam, it was impossible, even by taking immediate action toward its construction, to complete it in time to afford the desired relief in the water supply. They, therefore, ordered the construction of the Carmel Dams (Reservoir D) and the Titicus Dam (Reservoir M), as well as the completion of the Sodom Dam System and Reservoirs, which included two dams, two reservoirs and a connecting tunnel. The construction of these works was started as soon as practicable, and further investigations were authorized in relation to the proposed large dam, in order to determine whether the best available site had been found.

To this end an extensive series of diamond-drill borings was made along the valley of the Croton River from the site of Old Croton Dam to a point nearly at the mouth of the river, about 1 mile below the old Quaker Bridge site. The result was the decision of the Commissioners, in January, 1891, to build the large dam at the Cornell site, a point about 1¼ miles above Quaker Bridge, and so situated as to store nearly as much water as would have been stored by the Quaker Bridge Dam. The amount of storage by the dam if built at the Quaker Bridge site is estimated at 32 000 000 000 galls.; at the Cornell site, 30 000 000 000 galls.

In connection with these new dams and storage reservoirs are various older dams and natural lakes, throughout the water-shed of the Croton, which have been in use for the city's water supply for many

years, in connection with the Old Aqueduct; and the total storage capacity, upon the completion of the New Croton Dam, will be as follows:

Total storage in connection with the old works, including Central Park, Boyds Corners and Middle Branch Reservoirs..	9 541 000 000	galls.
Amawalk Dam............................	7 000 000 000*	"
Reservoir I, Sodom and Bog Brook Reservoirs..................................	9 028 000 000	"
Reservoir D, Carmel.......................	9 000 000 000*	"
Reservoir M, Titicus......................	7 167 000 000	"
New Croton Dam Reservoir................	30 000 000 000*	"
Jerome Park Reservoir....................	1 500 000 000	"
	73 236 000 000	"

As the large reservoirs within the city territory cannot be emptied below certain limits without impairing the supply, the available storage capacity may be stated as about 70 000 000 000 galls.†

The construction of the New Croton Dam was begun in October, 1892, the contract for its construction having been let the preceding August. At the present time it is about two-thirds completed, and, as a general description of the structure, embodying its main features, is essential to the purposes of this paper, the following extracts from the "Report of the Chief Engineer, A. Fteley," Past-President, Am. Soc. C. E., "to the Aqueduct Commissioners, 1887 to 1895," are reprinted here, as they seem to embody the main points and important features in comparatively few words.

"The New Croton Dam at Cornell Site which is to form the largest reservoir of the system, on the lower part of the Croton River, was begun in October, 1892. It is located about 3¼ miles above the junction of the Croton with the Hudson, and about 1 mile above Old Quaker Bridge. The course of the Croton at this point is approximately west.

"At the dam location, rock (gneiss) crops out at the surface on the north side of the river, rising with a steep slope, which terminates at the top of a hill about 300 ft. high. The bed-rock on the north side dips quickly just before reaching the bank, and soundings show it at about 75 ft. below the river-bed. At this point, on a line about

* Approximate.

† Report of the Chief Engineer to the Aqueduct Commissioners, 1887-1895, p. 82.

parallel to and under the river, the rock changes abruptly from gneiss to limestone, with no marked change of surface level. The limestone extends across the valley at about the depth noted above, with some deeper pockets, and then rises gradually on the south side with the earth slope and below it, at varying depths, to a depth of 20 ft. at the extreme south end of the dam location.

"Under the river-bed the material above the bed-rock is largely sand, gravel and boulders. Approaching the south side of the valley, very compact hardpan and gravel next to the rock is indicated. The hardpan is surmounted next to the surface by a considerable layer of sand at the steep part of the slope. Along this slope, at about elevation 153 runs the Old Croton Aqueduct. The total distance across the valley at flow-line (elevation 200) is about 1 300 ft.

"The general features of the dam may be noted as follows:

"*An overflow, or spillway,* on the rocky side-hill forming the north slope of the valley.

"*A masonry dam* built on bed-rock and extending from its junction with the overflow at about the foot of the north slope of the valley, across and well into the south slope, where it ends in a wing-wall and core-wall for the embankment.

"*An embankment* with a core-wall extending to bed-rock from the end of the masonry dam up and along the south slope until elevation 220, the proposed top of this part of the dam, is reached.

"*The overflow* varies in height from 150 ft. at its junction with the main dam to about 10 ft., where it joins the side-hill at the upper end. This overflow runs along the side-hill and nearly parallel to the slope contours, curving up-stream at its junction with the masonry dam. The down-stream face of the overflow is to be formed in steps. From the spillway the water is to fall into a channel cut into the rock of the side-hill, through which the water will pass on its way to the river-bed below the dam. This overflow channel is to be about 50 ft. wide at the upper end and 125 ft. wide next to the main dam. The length of the overflow will be nearly 1 000 ft., elevation of top, 196.

"*The masonry dam* will extend from bed-rock to elevation 210, and provision is made for a roadway on top, 18 ft. in width. At the north end, near its junction with the overflow, is to be a gate-house of three chambers. Grooves in the masonry of the up-stream face will be provided for stop-planks, and in each chamber will be gates worked from the top of the dam, connecting with a 48-in. pipe. The pipes will extend through the dam, ending in a vault, containing stop-cocks to further control the flow of water. It is expected to place the lower openings in the gate-chambers at about elevation 75, nearly 30 ft. above the original river-bed, and to fill in this interval with earth, forming an embankment with a flat slope above the restored original surface, on the up-stream side.

"The masonry dam will be about 600 ft.* in length from its junction with the overflow to the back of the wing-wall at the south end, and its extreme height will be 260 ft. or more, as the soundings show some large and deep depressions in the rock surface below. Maximum thickness at bottom next to rock, about 190 ft.

"The embankment extending south from the wing-wall end of the masonry dam will have a core-wall extending throughout its length, founded on bed-rock, thus forming with the overflow and main dam a continuous masonry connection with bed-rock throughout the whole length. From elevation 64 down to bed-rock this wall is to be not less than 18 ft. in thickness; from elevations 64 to 200 the wall gradually diminishes to 6 ft. in width at the top. The elevation of the top of the embankment is 220; width at top, 30 ft. Up-stream slope, 2 to 1, paved; down-stream slope, 2 to 1, broken with three berms, each 5 ft. wide at different elevations. These berms will be ditched and paved to carry rain-water from the slopes, which are to be soiled and sodded.

"The Old Aqueduct is discontinued between the slope lines of the embankments, and is being replaced by a new section built on a curved line into the side-hill, nearer the extreme south end of the dam. At the junction of this new line of Aqueduct with the core-wall masonry, a second gate-house will be built for the purpose of connecting the water impounded in the New Reservoir with the Old Aqueduct.

* * * * * * *

"The gate-house foundation rests on bed-rock, and the curved line of the new section of the Aqueduct was designed to avoid the deep excavation for this foundation, which would have been necessary had the original location on the Old Aqueduct line been adhered to. The gate-house is drained by a system of 12-in. pipes, which are connected with the bottom of each chamber and unite into one pipe laid under the invert of that part of the new section of the Aqueduct lying on the down-stream side of the core-wall. Near the junction of the New Aqueduct Section with the Old Aqueduct, this drain pipe, after a short turn, emerges in the adjacent hillside.

"The center of the overflow and masonry dam, the core-wall, the gate-house foundations, the side walls of the Aqueduct, the backing of the gate-house chambers and inlet conduits will be built of rubble masonry. The overflow will be faced above the surface of the ground with coursed facing-stones cut to specified rises. On the down-stream side the steps are to be laid with block-stone masonry generally heavier in rise and width than the facing-stone, and of depth sufficient for a bond under the next step above.

* * * * * * *

* This length has since been increased to 710 ft.

"The main dam and the outer faces of its gate-house will be faced, wherever exposed, with facing-stone of the same class as that in the overflow.

* * * * * * *

"For the protection of the deep earth excavation, which is to take place in the bottom of the valley for the foundation of the dam, the river is diverted from its bed for a distance of over 1 100 ft. For that purpose an extensive rock cut has been made into the north side-hill and the river has been turned into this new channel" (125 ft. in width) "which is formed on the river side by a substantial river-wall founded in rock.

"This wall, parallel with the old river-bed and 600 ft. long, is connected at both ends with temporary wing-dams extending across the valley, above and below the site of the dam, thus making a complete inclosure, inside of which the excavation can take place without interference from the river. The wing-dams are built of earth with a timber core formed of two thicknesses of plank tongued and grooved, each 3 ins. in thickness. The timber core extends to a depth of 20 to 25 ft. below the natural ground. The toe of the dams on the river side is formed by heavy crib-work, intended to break the force of the current in time of freshet. The toe of the lower wing-dam is further protected by sheet piling and by a heavy weight of rock to counteract the erosive action of the large flow which may be discharged from the new channel into the river in case of a heavy freshet.

* * * * * * * *

"The total length of the protective work just described, including the river-wall and the wing-dams, is 1 600 ft. The capacity of the new channel has been designed to safely accommodate a flow equal to the largest freshet recorded in Croton River since the construction of the old works, when the discharge was approximately 15 000 cu. ft. per second."

In connection with this description, attention is called to Plate XXXV, which is an outline plan of the structure and shows, in addition to the various features noted above, the outline of the excavation necessary for the main dam foundation masonry, and the embankment to be built against the core-wall with which it forms the south end of the structure.

Figs. 1 and 2 show various sections of core-wall and embankment, of the main masonry dam at various points and the maximum section of the overflow wall where it crosses the temporary river channel.

The dam was designed and its construction is being superintended by Mr. Fteley, the Chief Engineer. He was assisted, for the mathematical computations necessary for determining the main section, by

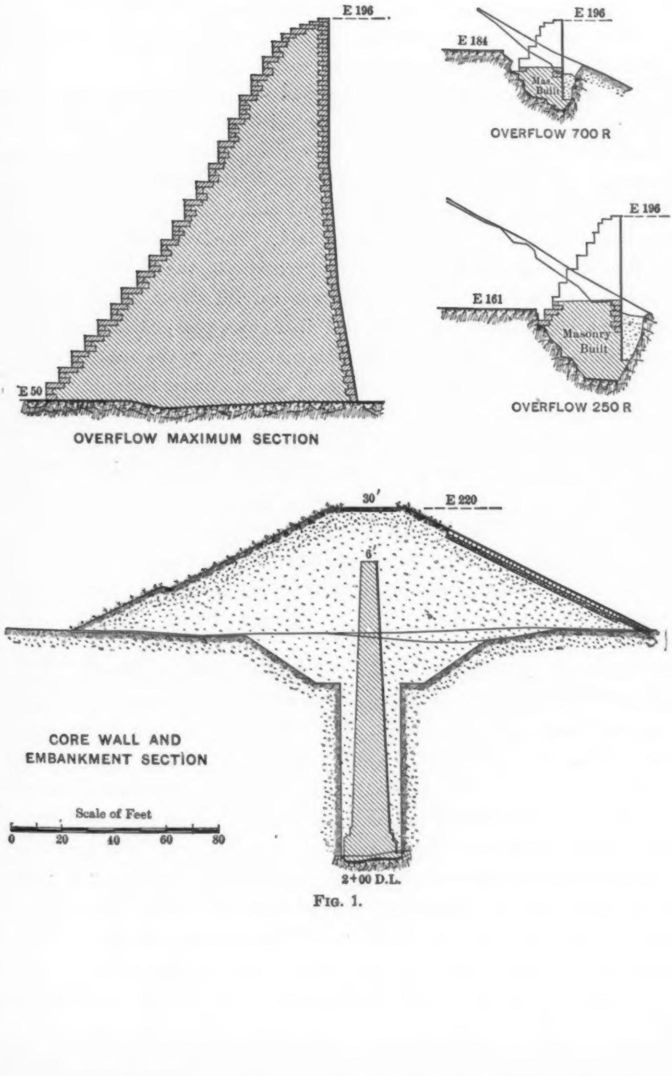

E 196
E 50
OVERFLOW MAXIMUM SECTION
E 196
E 184
Mas. Built
OVERFLOW 700 R
E 196
E 161
Masonry Built
OVERFLOW 250 R
30′
E 220
6′
CORE WALL AND EMBANKMENT SECTION
Scale of Feet
0
20
40
60
80
2+00 D.L.

FIG. 1.

E. Wegmann, M. Am. Soc. C. E., who has since developed and formulated the methods followed, in his book on high masonry dams.*

It may be said that the section adopted affords a factor of safety of 2 against any tendency toward the overturning of the structure.

The work of construction has been conducted, from its inception, under the immediate direction of the writer.

Since the foregoing description was written, the protective work has been completed, substantially as outlined. The earth and rock excavations for the foundations have been finished; the foundation masonry practically all laid, excepting a short stretch of the overflow which is to cross the river-channel cut and join the long stretch of overfall foundation masonry already laid. The length of this remaining stretch is about 250 ft. In the progress of the above work the section of the main dam masonry was carried about 110 ft. further into the side-hill at the south end than was planned at first, thus shortening the core-wall and embankment section by the same distance, and, owing to the rise in the bed-rock surface under the south slope, decreasing the maximum depth or height of the core-wall and embankment considerably from that of the original design.

Owing to the character of the limestone, which rendered deep excavation necessary at certain points, the extreme height of the masonry dam will range from Elevation—80, the lowest point reached in the foundation excavation, to Elevation 210, a total of 290 ft. For the same reason the extreme thickness of the main dam masonry at the toe is about 200 ft.

Borings.

The final location of the New Croton Dam resulted from the indications furnished by an extensive series of diamond-drill borings, during which the Croton Valley was explored thoroughly along the line of the river from an old mill at the head of tide water, about three-quarters of a mile below Quaker Bridge, to Old Croton Dam, a distance of about 5 miles. The general system for determining upon the position of the borings proposed, was as follows: Whenever the appearance of the surface seemed to be favorable a number of drill holes were made on a line parallel with the river, and, if one of them gave indication of the proximity of bed-rock to the surface, a transverse line of holes was drilled across the valley at that point.

* "High Masonry Dams," by E. Wegmann, Jr., M. Am. Soc. C. E., New York, John Wiley & Sons, 1891.

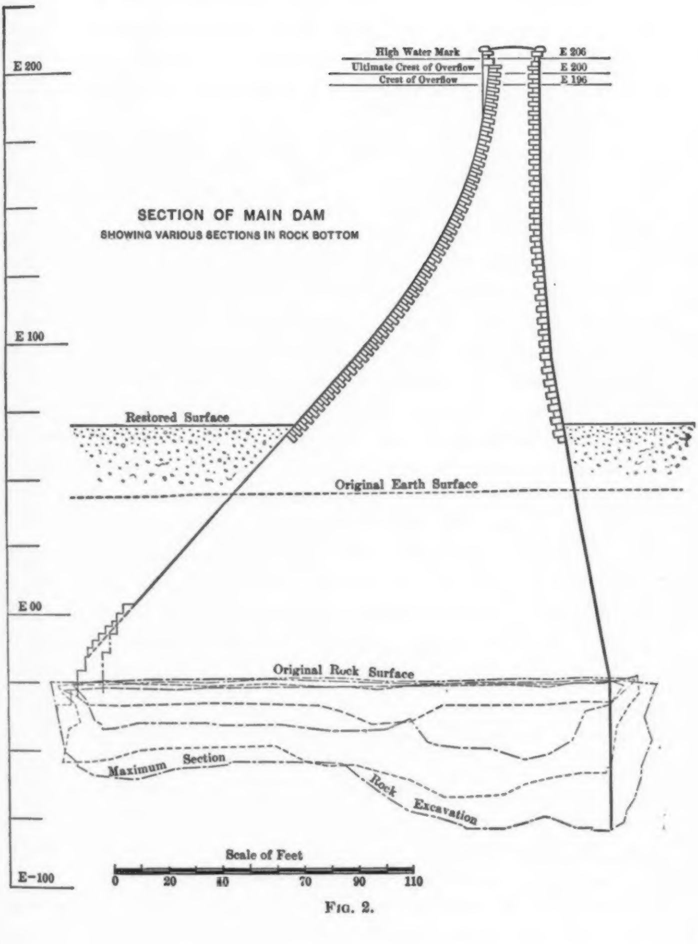

SECTION OF MAIN DAM
SHOWING VARIOUS SECTIONS IN ROCK BOTTOM
High Water Mark
Ultimate Crest of Overflow
Crest of Overflow
E 206
E 200
E 196
E 200
E 100
E 00
E-100
Restored Surface
Original Earth Surface
Original Rock Surface
Maximum Section
Rock Excavation
Scale of Feet
0
20
40
70
90
110

Fig. 2.

In this way a large number of transverse lines was drilled, and it was found almost invariably that wherever the bed-rock cropped to the surface on one side of the valley it dipped down sharply on the other side to a depth at which, in most cases, it would be impracticable to establish a foundation. As a rule, gneiss was found, but at various points on either side there are formations of limestone with clearly defined points of separation which, in some cases, were under the bed of the stream.

Several of the more favorable locations thus indicated were explored more particularly by a number of transverse lines of holes about 100 ft. apart, and when the present established location was finally determined upon, additional borings were made, to cover the site of the masonry structure, at intervals of 50 ft. Fig. 3 shows the location and result of these borings, as well as the outline of the proposed foundations. It will be seen that there were in places indications of a considerable depth of soft white rock (partly disintegrated limestone), before the hard rock was reached, extending in one case to an extreme of nearly 40 ft. The holes drilled in the rock were, as a rule, 2 ins. in diameter, and were carried to a depth presumably sufficient to establish the character of the rock below. The hard white rock sought for, and, as a rule found before the borings ceased, was mostly bluish limestone, while the soft white rock varied in its texture from white limestone, friable under some pressure, to very friable or wholly decomposed rock. The line of separation between the limestone and the gneiss was shown to be directly under and parallel to the river-bed. The borings indicated further, the presence of seams, more or less open, in the limestone, and the frequent reports of the sudden loss of the water (*i. e.*, the water supplied by the steam pump to wash out the holes as the borings progressed) showed that these seams were connected in places with rather free flowing outlets. As the general level of the bed-rock was at Elevation —25, or about 75 ft. below the river, and as the water table in the sand and gravel above this bed-rock was substantially the same as the river level, it is perhaps a question of some interest as to how and where this disappearing stream went, and, in case of its reappearance, what were the causes which may have led to it. Copies of the drill runner's log, which follow, show the records of Holes Nos. 95 and 99.

SCALE OF FEET
0 100 200 300 400

FIG. 8.

PLAN OF MAIN DAM EXCAVATION
SHOWING SOUNDINGS

These furnish two illustrations out of a number of cases in which the water disappeared and reappeared after an interval. Hole No. 99 is especially noticeable, as the final disappearance of the water did not occur until the drill had reached its lowest level, Elevation —76.80.

HOLE No. 95.—Elevation of Jack Plank........... 71.9
" " Ground.............. 69.6

Date. 1892.	Material.*	Depth, below Jack Plank.	Remarks.
April 20.	S. & B.	7.23	
" 21.	"	27.50	
" 22.	....	....	Broke casing; third joint up 28.35.
" 22.	....	....	Pulled out; got back to 28.35.
" 22.	Boulder.	28.35	X bit 10 ins. below shoe.
" 22.	C. S. & B.	30.00	
" 25.	"	36.00	Don't stand up; fills in.
" 26.	"	43.25	" " "
" 27.	"	51.00	" " "
" 28.	"	56.00	" " "
" 28.	S. G. S.	58.00	Very little flow as soon as X bit is below shoe.
" 29.	"	59.00	Very little flow as soon as X bit is below shoe.
" 29.	H. S. & S.	74.00	Stands up and fills in; can pound down; stands up; no flow.
" 30.	"	79.00	Stands up and fills in; can pound down; stands up; no flow.
May 2.	"	91.00	Stands up and fills in; can pound down; stands up; no flow.
" 3.	"	94.00	Fills in very bad; cannot get powder down.
" 4.	"	98.67	Stands up good.
" 5.	"	100.87	—28.97 top of soft white rock.
" 5.	S. W. R. & Sand.	104.85	—32.95 I think this is fine sand; the floor was clear.
" 5.	S. W. R.	105.90	No core.
" 6.	"	106.90	Lost flow.
" 6.	"	110.00	Flow came back; no core.
" 6.	"	111.00	The rock is a little harder; no core.
" 6.	"	114.85	Not hard enough to core.
" 6.	"	118.85	" " " yet; no core.
" 6.	"	122.00	Lost flow.
" 6	"	122.95	Not hard enough to core; no core.
" 6.	"	123.30	Commenced to core —51.40.
" 6.	H. W. R.	124.55	0.90.
" 6.	"	126.95	0.90.
" 6.	"	130.25	1.80 Elevation of water in casing x 45.9.

* S. & B. —Sand and boulders.
C. S. & B.—Coarse sand and boulders.
S. G. S. —Sand, gravel, stones.
H. S. & S.—Hard sand and stones.
S. W. R. —Soft white rock.
H. W. R. —Hard white rock.

"This hole is the same as Hole No. 88; stands up very good, but could not go far below the shoe, the flow would go away. The rock from elevation —28.97 —51.40 was very soft, but stood up very good, and did not cave in, if it had I could not have drilled so far down. W. J. S. (Signed) Tierney, Foreman."

HOLE No. 99.—Elevation of Jack Plank............72.2
" " Ground...............70.0

Date. 1892.		Material.*	Depth, below Jack Plank.	Remarks.
May	19.	S. & S.	15.00	Moved, set up, down to 15.00.
"	20.	F. S. & S.	36.50	Loose fine sand and no flow.
"	20.	C. S. & S.	42.86	Stands up good, flow came back.
"	20.	"	51.00	" " "
"	23.	"	53.00	" " "
"	23.	S. G. & S.	64.50	Telescoped with 4-in. casing to 38.50.
"	24.	"	73.00	" 2½-in. "
"	25.	"	79.00	Fills in bad.
"	26.	"	88.00	"
"	27.	"	91.50	"
"	28.	"	95.00	Stands up good, very stony.
"	30.	"	97.50	" "
June	1.	"	99.58	Top of S. W. R. —27.38.
"	1.	S. W. R.	100.80	
"	1.	"	103.0	Put in diamond bit.
"	1.	"	104.9	No core.
"	1.	"	108.4	"
"	2.	"	114.1	"
"	2.	"	121.8	"
"	2.	"	125.9	"
"	2.	"	128.2	"
"	2.	"	130.5	Commenced to core —58.30.
"	2.	H. W. R.	131.6	0.60 core.
"	3.	S. W. R.	133.6	No core.
"	3.	"	137.50	No core. (—65.3).
"	3.	H. W. R.	140.40	1.70 core, commenced to core —68.2.
"	3.	"	142.00	0.60 "
"	3.	"	143.50	0.40 "
"	6.	"	145.50	0.25 "
"	6.	"	147.50	0.60 " Lost part flow 147.8.
"	6.	"	148.90	0.75 "
"	6.	"	150.15	0.60 " Lost all flow 149.0.
				"M. TIERNEY."

*S. & S. —Sand and stones.
F. S. & S.—Fine sand and stones.
C. S. & S.—Coarse sand and stones.
S. G. & S.—Coarse gravel and stones.
S. W. R. —Soft white rock.
H. W. R. —Hard white rock.

"June 2d. Put in diamond bit at 103.0. Commenced to core at 130.50; rock was not soft like mush; could not turn rods down with the tongs, but was not hard enough to core; did not find any seams or soft spots; stood up good; did not fill in. June 3d, no seams, no soft spots, but not hard enough to core. X Rock in Hole 99 was hard enough to stand up but did not core. Did not find any soft mushy seams. Commenced to core —58.3, cored to —59.40, hard did not core until I got to —68.20. Then I picked up some core all the way down, as report will show, lost part flow —75.60. Lost all flow —76.80. W. J Sager."

Figs. 4, 5 and 6 are three sections of the foundation rock on which the main dam is built. The limits of hard and soft rock surface, as indicated by the soundings, as well as the actual surface exposed upon excavation and the actual surface built upon, are shown. These sections are interesting as a comparison between the possible results, as

shown by the diamond-drill work, and the actual results obtained. In a general way, it may be said that the rock was found to be more broken up and traversed by seams, fissures and soft streaks, in all the various conditions exhibited by limestone ledges, than might have been expected from general surface indications in the neighborhood and from the borings themselves. To a certain extent, the same was true of the gneiss, the surface of which was found to be full of slips and seams running in every direction between hard masses, while extensive pockets and seams of disintegrated rock of considerable

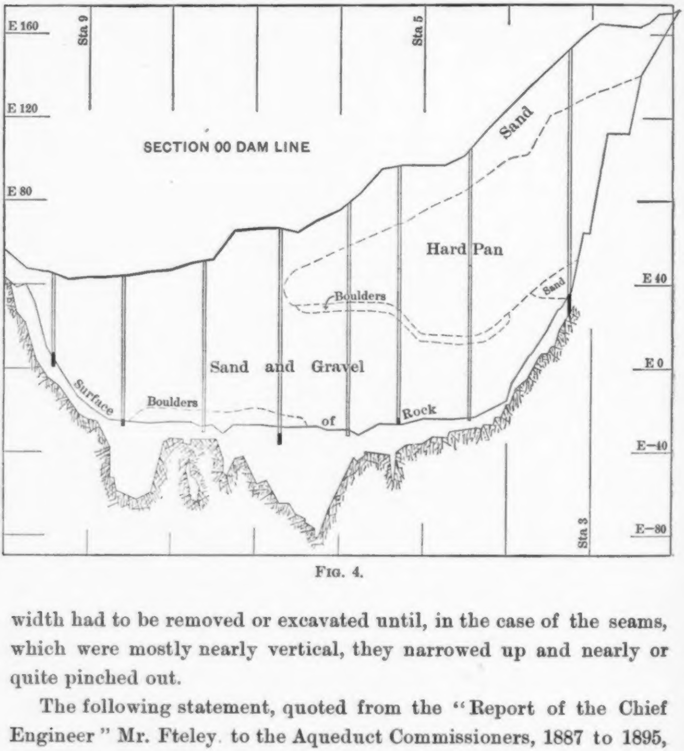


FIG. 4.

width had to be removed or excavated until, in the case of the seams, which were mostly nearly vertical, they narrowed up and nearly or quite pinched out.

The following statement, quoted from the "Report of the Chief Engineer" Mr. Fteley to the Aqueduct Commissioners, 1887 to 1895, is given here in explanation of the fact that it was finally decided to build the dam at this point, although at the time the decision was made all facts in connection with this location had not been devel-

oped, and its superiority to other sites was still an open question, while the additional borings, made, as previously noted, after the site had been decided upon, showed no more encouraging results at least than those made earlier.

"No very favorable location was found, and the writer reported to the Aqueduct Commission on October 8th, 1890, that it would be advisable to abandon for the present the Quaker Bridge site, and to build a dam of less magnitude a short distance below the present Croton Dam (see Location 2, Line C, on Sheets 27 and 29). Although the reservoir to be thus formed would have contained an available

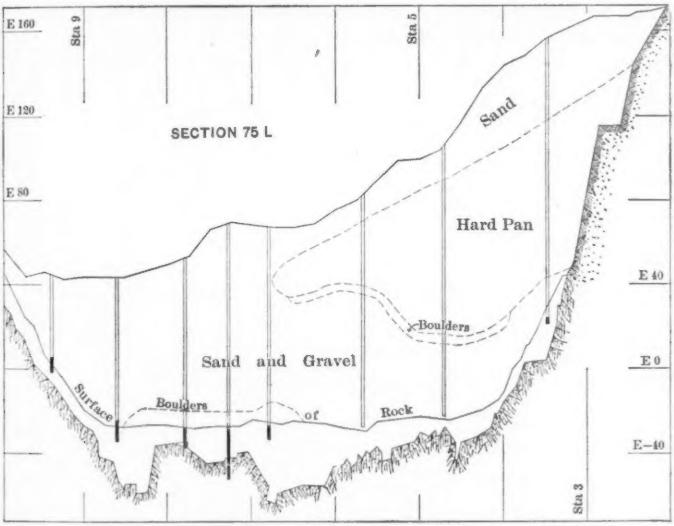


FIG. 5.

storage of about one-half that of the Quaker Bridge Reservoir, the principal reasons given in favor of that opinion were:

"*First.*—That the storage to be thus obtained would be sufficient for many years to come.

"*Second.*—That the height and cost of that dam would be much less, and that it could be built in less time.

"*Third.*—That the experience which would soon be acquired of the effect of the large storage reservoirs under construction on the quality of the water, would better enable the authorities in charge to ascertain whether it would be of good policy in the future to build the higher dam or to resort to some other mode of increasing the supply.

"*Fourth.*—That the interest of the money thus saved for the present would, after twenty-five years, represent a large part of the money necessary to then build the higher dam, with the result that the city would then have two dams instead of one for nearly the same expenditure.

"The report also mentioned that another site (the Cornell's site), not then fully explored, presented good features and should be further considered.

"The Aqueduct Commissioners voted to adopt the last-mentioned site, which is one mile and a quarter above Quaker Bridge.

"Borings made subsequently to this decision disclosed that the rock strata, at places, were found to be at a greater depth than was anticipated; hence, the excavation will be deeper than was originally intended, and the bulk of masonry will be correspondingly larger."

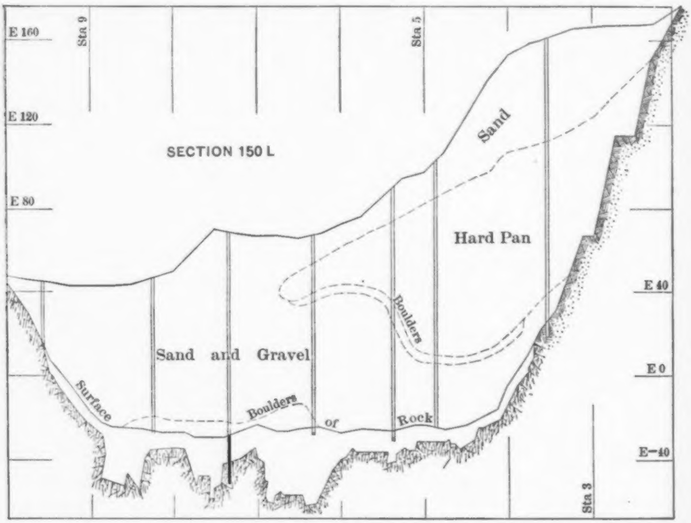
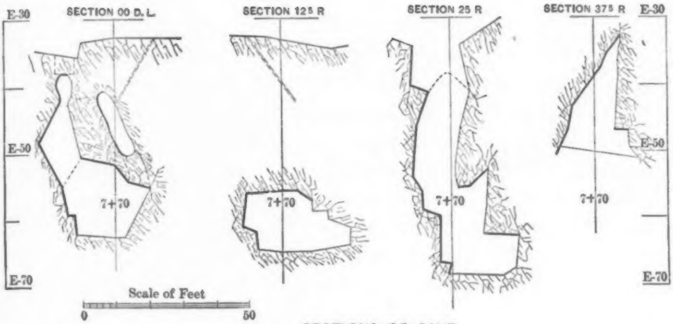


Fig. 6.

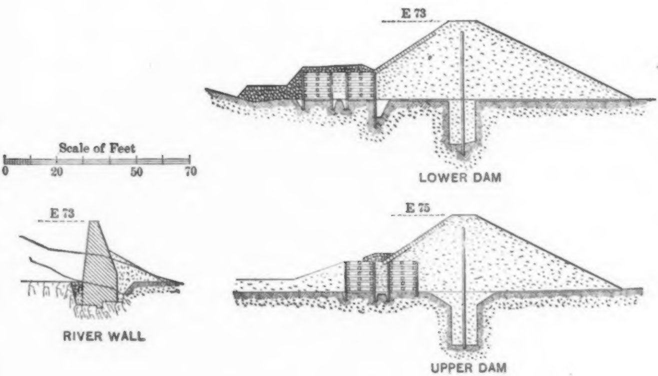
Protective Work.

Plate XXXV shows the general plan of the protective work designed and built for the purpose of enabling the deep excavation necessary for the main dam foundations to be carried on with the smallest chance of interruption from floods.

The section of the river wall which separates the new river channel from the main excavation is shown by Fig. 7. This wall is 600 ft.



SECTIONS OF CAVE



SECTIONS OF WING DAMS

Fig. 7.

long, and is built on the underlying gneiss, care being taken, throughout its length, to make its bond with the foundation rock as complete as possible, particularly throughout that portion which comes within and forms a part of the main dam. Throughout this section, excavation was made in the foundation rock to a considerable depth to get below open seams and fissures, and during its construction a considerable portion of the foundation of the main dam overlying the channel cut was laid up to the grade of the channel, advantage being taken of the necessary foundation work of the adjacent river wall to do it.

Fig. 7 shows also sections of the upper and lower earth wing-dams as built; and their position, relative to the main excavation cut, is shown in Plate XXXV. The main lines of 3-in. sheeting, which were relied upon as the water stops, were carried down from 20 to 25 ft. below the original surface. As most of the material in which this sheeting was placed was coarse, loose gravel and sand, resort was had to trenching, with sides temporarily sheeted, and the permanent sheeting, after being placed in position, was driven by heavy hand mallets down an additional foot or two. At the east end of the upper wing-dam, however, for a considerable distance, the bottom was found to be of quicksand, and the sheeting was put down, through a considerable part of the depth reached, by means of a water-jet and heavy hand mallet.

The crib-work is designed to protect the embankment toes from the great erosion to be expected in case of a heavy freshet, while the extra sheeting and loading of stone on the lower wing-dam crib is a still further protection against the wash of the discharging channel, which, in extreme cases, might be strong enough to displace the loading and possibly cause a slight movement of the cribs which, in such cases, are so planned as to yield measurably outward without materially endangering the toe of the embankment.

While the river channel and these dams are designed to carry in emergency 22 ft. of water, or more, it may be said that at the present writing the deepest flow experienced through the channel has been about 11 ft. This was due to a warm rain of 3.6 ins., most of which fell in about 12 hours, on 3 ins. of snow lying on deeply frozen ground, in the month of February. From this it is easy to see that a combination of circumstances resulting in a flow which would tax the channel to its full capacity is quite possible.

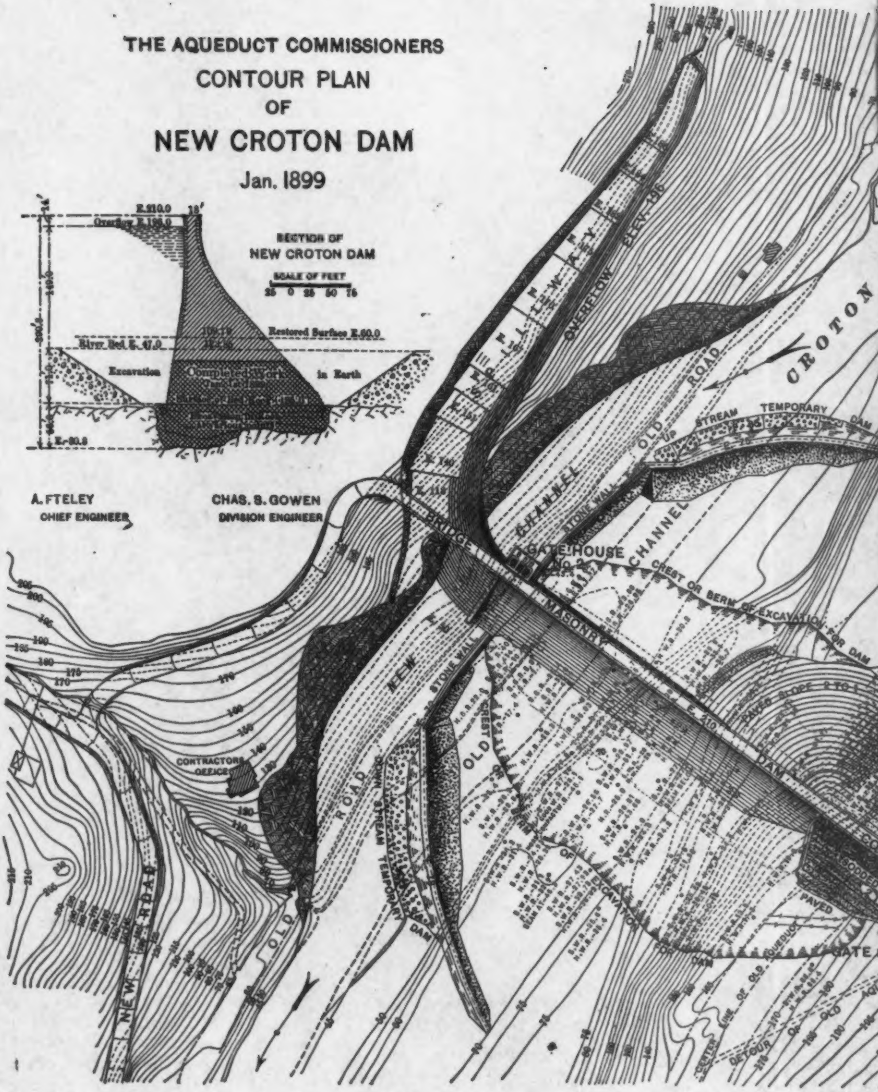
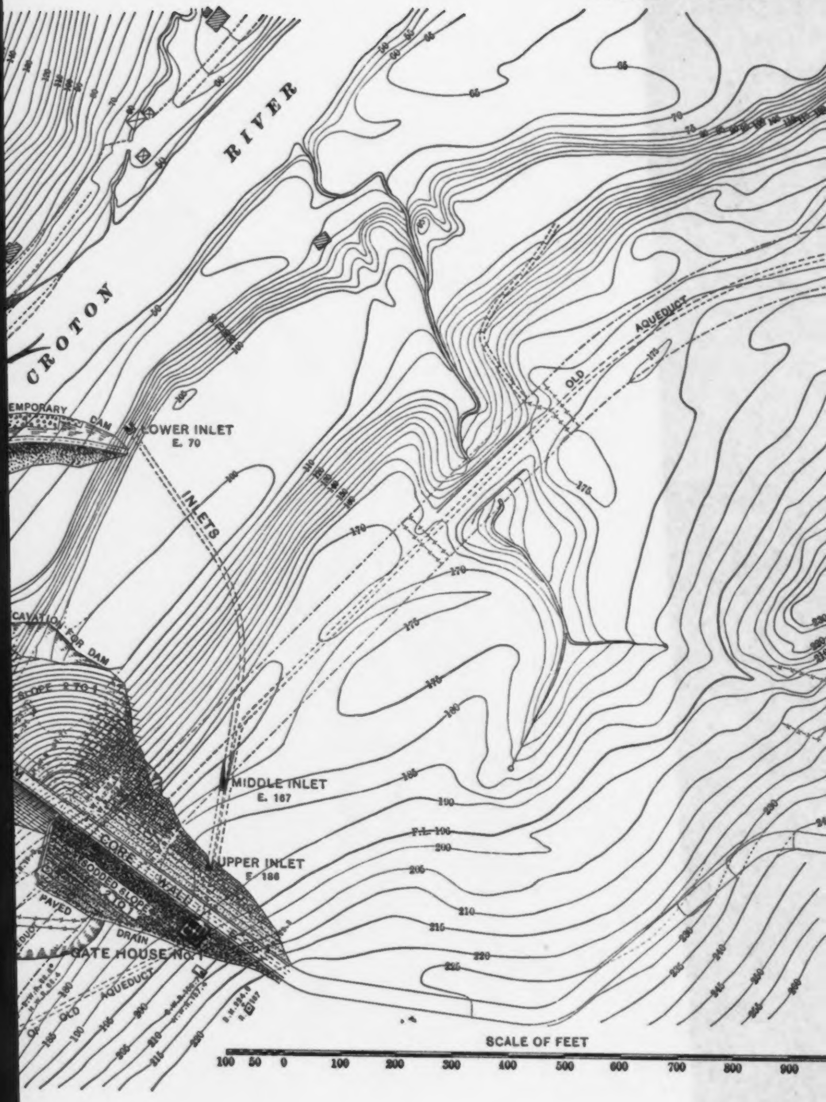

THE AQUEDUCT COMMISSIONERS
CONTOUR PLAN
OF
NEW CROTON DAM
Jan. 1899
SECTION OF
NEW CROTON DAM
SCALE OF FEET
Restored Surface E.60.0
Excavation
in Earth
Completed Work
A. FTELEY
CHIEF ENGINEER
CHAS. S. GOWEN
DIVISION ENGINEER
CROTON
TEMPORARY
UP STREAM
OLD ROAD
CHANNEL
OVERFLOW
GATE HOUSE
CREST OR BERM OF EXCAVATION FOR DAM
MASONRY
DAM
BRIDGE
STONE WALL
NEW ROAD
OLD ROAD
CONTRACTORS OFFICE
DOWN STREAM TEMPORARY DAM
PAVED SLOPE
PAVED
GATE HO
DETOUR
AQUEDUCT

PLATE XXXV.
TRANS. AM. SOC. CIV.
VOL. XLIII, No. 8
GOWEN ON FOUNDATIONS OF N



LATE XXXV.
M. SOC. CIV. ENGRS.
XLIII, No. 875.
TIONS OF NEW CROTON DAM.

Before leaving the subject of the protective work, attention is called to the somewhat extensive and perhaps seemingly permanent character of its design and construction. This work involved, in the construction of the river channel, an earth excavation of about 100 000 cu. yds., and rock excavation of about 106 000 cu. yds. The river wall and wing-dams include in their construction:

Earth Excavation	8 500 cu. yds.
Vertical Trench Excavation	6 700 " "
Refilling and Embankment	58 000 " "
Rock Excavation	4 000 " "
Timber	390 000 ft. B. M.
Crib Work	7 000 cu. yds.
Rubble Masonry	10 000 " "
Paving and Rip Rap	2 000 " "

While the cost of the above work is a large amount (upwards of $350 000), its proportion to the total cost of the dam, which may amount to $5 000 000, is not excessive, and it must be remarked that a considerable portion of it will form a part of the permanent structure. It seems to have been justified on account of the very efficient protection it has afforded to the extensive excavation work, both of earth and rock, and the foundation masonry work, which have been carried along steadily for three years and which, in the case of the masonry and refilling, must continue for another year at least before the dams will cease to be necessary. The extreme depth of the pit, in which the work has been done, below the river bed, is 130 ft.

Earth Excavation, Main Dam Foundation.

This work involved preparation for a foundation on rock extending from about Station 3 + 30 to about Station 10 + 00, where the new river channel, formed in connection with the protective work, is merged into the foundation, and which varies in width from about 200 ft., at the lowest point, to about 130 ft. at Station 10 + 00 and 140 ft. at Station 3 + 30, on the line of the back of the proposed wing-wall (see Fig. 3). The necessary earth excavation covering this area was about 885 000 cu. yds., consisting largely of loose sand, gravel and boulders with, however, at the south end of the pit, a large area of hardpan excavation, this hardpan forming, to a considerable extent,

the slopes at this end of the excavation, and extending in depth at the extreme south end, *i. e.*, the point of junction of the main dam masonry section with the core-wall, from the surface of the bed rock to about Elevation 130, above which it was surmounted by loose, fine sand reaching to the surface. Figs. 4, 5 and 6 are sections indicating at various points the relative positions of the different kinds of earthy material which had to be moved, and the south end slope lines to which the excavation was made. In the case of the gravel and sand, the slopes were $1\frac{1}{2}$ horizontal to 1 vertical, and in the hardpan at the extreme south end $\frac{1}{2}$ horizontal to 1 vertical, with a berm about half-way up the slope; while on the quarters, where the depth was considerably greater, the slopes and berms were varied somewhat, as the end slopes were merged into the side slopes. In laying out the slopes, consideration also had to be given to the fact that on the quarters, at a comparatively low elevation, the hardpan was underlaid with layers of boulders and gravel which extended to the bed rock as it dipped in its surface between Station 3 + 30 and Station 5 + 00, where it reached the general level of the rock in the valley bottom. These earth slopes were all planned to allow for a toe berm of 20 ft. in width, at the rock surface, and this space proved to be necessary and useful in further operations in the rock bottom below.

The slopes stood very satisfactorily, on the whole, no particular trouble resulting from washing or sloughing, in case of the gravel slopes, so long as surface drainage outside the pit was properly controlled. In the case of the hardpan, steep slopes which in combination with the sand above and at certain points with sand, gravel and boulders below, were, at the maximum, 150 ft. in height, the only trouble experienced was during the open winter of 1897–98, when successive freezings and thawings caused the slope surface to slough off in successive thin layers representing in thickness the depth in each case to which the frost had penetrated since the preceding thaw.

The maximum width of the pit, from the top of the up-stream slope to the top of the down-stream slope, was about 600 ft., and the largest area of cross-section excavation, above bed rock and parallel to the trend of the valley, *i. e.*, at right angles to the dam line, was about 49 000 sq. ft.

Figs. 3, 4, 5 and 6 show in plan and section the crest and toe lines of the slopes and the location and elevation of the berms in the steep

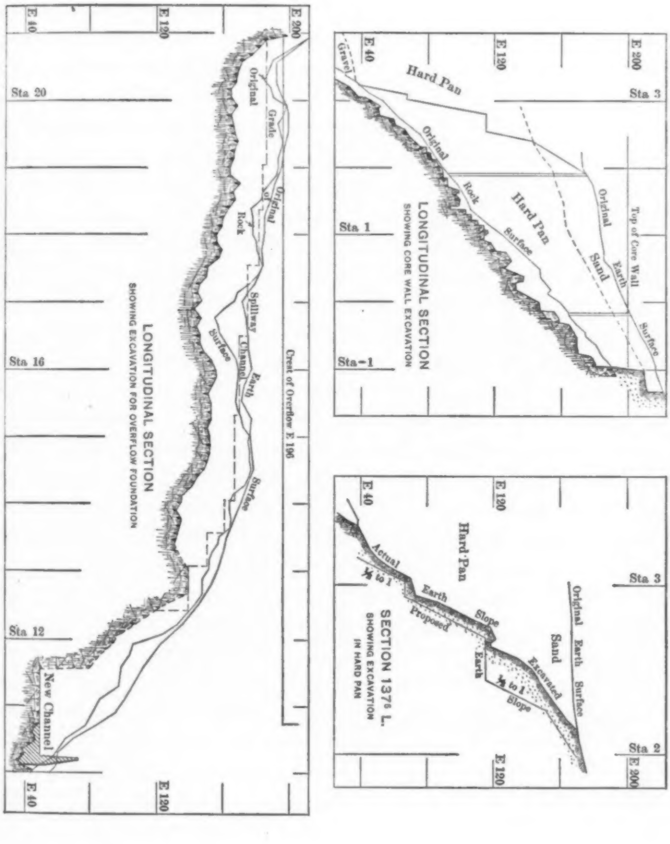

LONGITUDINAL SECTION
SHOWING EXCAVATION FOR OVERFLOW FOUNDATION
Crest of Overflow E 196
Original Grade
Original Surface of Rock
Spillway Channel
Earth Surface
New Channel
Sta 20
Sta 16
Sta 12
E 40
E 120
E 200
LONGITUDINAL SECTION
SHOWING CORE WALL EXCAVATION
Gravel
Hard Pan
Original Rock Surface
Hard Pan
Sand
Original Earth Surface
Top of Core Wall
Sta 3
Sta 1
Sta -1
SECTION 137+6 L.
SHOWING EXCAVATION
IN HARD PAN
Hard Pan
Actual Earth Slope
Proposed
½ to 1
Sand
Original Earth Surface
Excavated
Earth
½ to 1
Slope
Sta 3
Sta 2

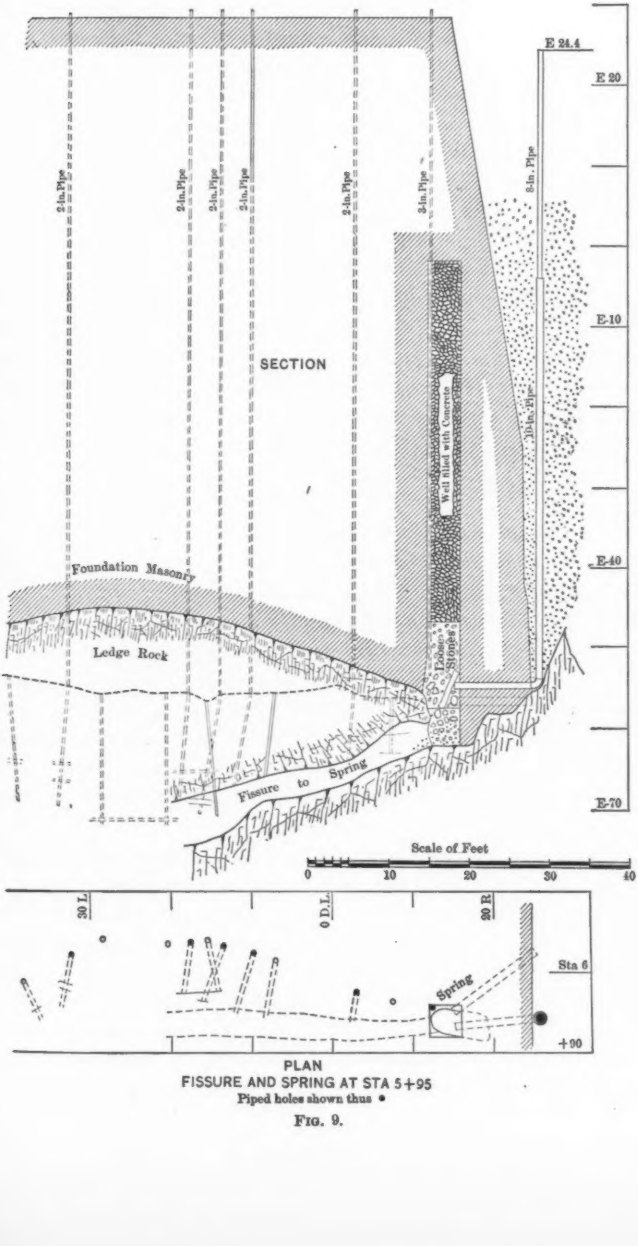
Fig. 8.

slope at the south end of the pit. The line of the masonry foundation is also indicated and its connection with the core-wall. A section at 137.5 L., in Fig. 8, shows the ordered and actually excavated slope in the hardpan at its highest point.

A very large amount of this excavation, lying on the south slope of the valley and above the level of the river, was removed by steam-shovels, three of which were in use at one time. The first work done in sinking below the river bed was by means of a large "orange peel" dredge, specially constructed for the purpose and used for the excavation of the loose gravel and sand until the near approach in depth to bed-rock, and the necessity of beginning rock excavation, demanded a change in methods, as the dredge work was dependent upon a certain depth of water in which to work the bucket efficiently, while the rock excavation rendered close drainage necessary. For the further prosecution of this work resort was had for some time exclusively to three cable-ways stretched across the valley longitudinally along the line of the dam at such transverse intervals as to cover the plan of foundation. These cables were installed for the purpose of aiding the earth and rock excavation and, ultimately, for taking in stone and other material for the dam masonry. They were used for some time in connection with the dredge above mentioned, and were in turn supplemented, when the rock excavation work assumed large proportions and there was considerable earth work remaining, by railway inclines placed successively at different points on the side slopes and worked by means of stationary hoisting engines and cables.

With the use of railway inclines, steam-shovels were again operated, and a large amount of coarse indurated gravel, lying just above the bed rock at the north end of the main cut, was thus excavated, and, as the excavation progressed toward the south end of the cut and the hardpan was reached, it was removed almost wholly with the aid of heavy steam-shovels, although the slope trimming at the south end and on the quarters, and some bottom cleaning up on the rock surface, had to be done by hand with the aid of skips and derricks.

Fig. 1, Plate XXXVI, shows a part of the river wall and lower wing-dam, and the progress of the main dam excavation to September, 1895. The large pit shown was excavated mainly by means of the dredge, shown on the extreme right, with considerable assistance from the cableways, for which the material was excavated by hand into large



PLAN
FISSURE AND SPRING AT STA 5+95
Piped holes shown thus •

FIG. 9.

scale pans, and hoisted and transferred to the foot of the heavy slope shown in the rear, where it was dumped into cars. The levels above the pit, shown in Fig. 1, Plate XXXVI, were excavated with steam-shovels, and later, as further progress was made into the hard material of the great slope at the south end of the cut, steam-shovels were again placed at a lower level in the pit and the inclines were used as mentioned previously. Fig. 2, Plate XXXVI shows more particularly the steep slopes in the hardpan as finally shaped, and Fig. 1, Plate XLV shows in a more general way the side slopes, but at some time after the excavation was completed, when a small amount of back-filling had been done. Also, at this time, cuts had been made in the side slopes, forming berms on which side tracks were laid, to furnish supplies for the foundation masonry. Fig. 2, Plate XXXVI, shows particularly the hardpan slopes ($\frac{1}{2}$ horizontal to 1 vertical) and berms at the south end and on the quarters, as well as the underlying rock bottom excavated for the foundation masonry.

Core-Wall Excavation.

The core-wall extends from the south end of the main dam for a distance of about 430 ft. into the side hill. Its general section and the cross-section of the trench excavated for it are shown in Fig. 1. The maximum width and height of this wall, which occurs at its junction with the main dam masonry, are, respectively, 18 ft. and 175 ft. The material excavated for the wall was hardpan above the limestone foundation up to within a depth from the original surface varying from 24 ft. to 8 ft. Above this hardpan were gravel and sand. The general extent, as well as depth of excavation for this wall, together with the line limiting the top of the trench excavation, are shown on the profile, Fig. 8.

The trench walls were vertical, the sustaining power of the hardpan allowing the sheeting and bracing to be done after the completion of the successive levels excavated, which levels varied from 6 to 12 ft. in height or depth, according to the depth of the section of trench then under excavation. As stated, the hardpan, throughout the length of this trench, extended to the rock foundation, which showed considerable variation in hardness and texture, and called for excavation of considerable depths below the rock surface in certain places before compact layers of sufficient hardness were found. Fig. 1, Plate

FIG. 1.—FEBRUARY 7TH, 1896. VIEW FROM UPPER END OF FOOT-BRIDGE, LOOKING S. E. FLOOD IN FOREGROUND.

FIG. 2.—OCTOBER 1ST, 1897. SLOPE AND BERM AT SOUTH END OF CUT.

XXXVII, shows the rock bottom ready for the masonry of the core-wall, as well as the sheeting and sides of the trench for a certain distance up, at Station 1 + 80, 150 ft. from its junction with the main dam. At this point the rock was sufficiently compact and of necessary bearing strength, although not very hard, and the steps shown in the inclined surface of this foundation were made with picks and shovels. The depth to which the rock was excavated varied from 4 to 7 ft.

The width of the trench is measurably greater than the thickness of the core-wall, and the difference was liberally planned in order that there should be no difficulty in finding working room at the bottom of the trench to remove the bracing and sheeting after the masonry foundations of the wall were started. It also gave proper opportunity to place the refilling; which was of the same material as had been excavated, and was placed very carefully in layers varying from 2 to 4 ins. in thickness, and thoroughly rammed by hand. Advantage was also taken of this extra width to widen the footing or lower courses of the core-wall, thus increasing the bearing surface in certain places where the rock foundation might possibly call for it, and the section shown in Fig. 1 is taken at one of these points.

As to the thickness of this wall, which it will be noted is somewhat massive, varying from 6 ft. in thickness at the top to 18 ft. at the lowest point, it may be said that the wall was purposely designed not only as a water-tight screen reaching from bottom to surface between the upper and lower sections of the enclosing embankment, but also to afford a substantial resistance to any overturning or crack-producing force which might be caused in the course of time by the saturation of the up-stream bank and its consequent increase of unit weight.

The maximum depth of sheeted vertical trench excavation, including the depth of excavation in the foundation rock, was 75 ft. This point was at Station 2 + 50. At this point the top of the vertical trench was 27 ft. below the original surface of the ground. The earth material above the core-wall trench level was excavated by steam-shovel; below, in the trench proper, it was excavated by pick and shovel, and removed by derricks. Black powder was generally used in sinking the trench, at the lower levels particularly, to loosen the hardpan, and it was used very extensively for the same purpose in the main cut, both for facilitating the work of the steam-shovels and for all handwork done in the removal of hardpan.

Overflow Excavation.

At the present writing, the completed overflow foundations, embracing a length of 750 ft., extend along the side hill on the north side of the river and finally abut in the rock of the hill at the upper end.

This rock foundation is entirely country rock, or gneiss, and the amount of superimposed earth was not large, and was mostly sandy loam on the surface, with underlying gravel.

Fig. 8 shows a profile of the earth and rock excavation as well, and on Fig. 1 are shown representative cross-sections, indicating more clearly the extent to which rock excavation was found necessary to insure a fairly tight bottom. The rock was full of seams and faults, and considerable depths had to be reached at certain points in order that open seams running across the line of the structure might be followed until they pinched out. The extensive rock excavation in the front of this foundation work, shown in the cross-sections, was necessary to provide the waterway leading from the spillway bottom to the old river-bed below the main dam.

Rock Excavation and Foundation for the Main Dam.

As stated in the general description of the dam, the rock on the north side of the valley, on the steep side hill, cropped out at points very near the surface. It was formed of gneiss, considerably fissured, but generally sound after reaching a certain depth in the ledge. This gneiss extended to the line of the old bed of the river, where its depth below the surface was much greater, being about 75 ft. The section under consideration was found to be well broken up near the surface by open seams of considerable width, varying from 2 to 3 ins. in cases. Such seams were filled with earth, and extended in all directions. There were also some strata of rock, more or less disintegrated. These varied from 1 to 3 ft. in width or thickness, and were removable with pick and shovel for some depth from the surface. The dip and strike of this rock were about the same as that of the limestone beyond; the dip being nearly vertical and the strike following the line of the valley at right angles to the dam.

The profiles, Figs. 4, 5 and 6, show the excavation necessary to get below the loose and open seams, which in places was considerable, as

FIG. 1.—MAY 27TH, 1897. STEPPING AT BOTTOM OF CORE WALL.

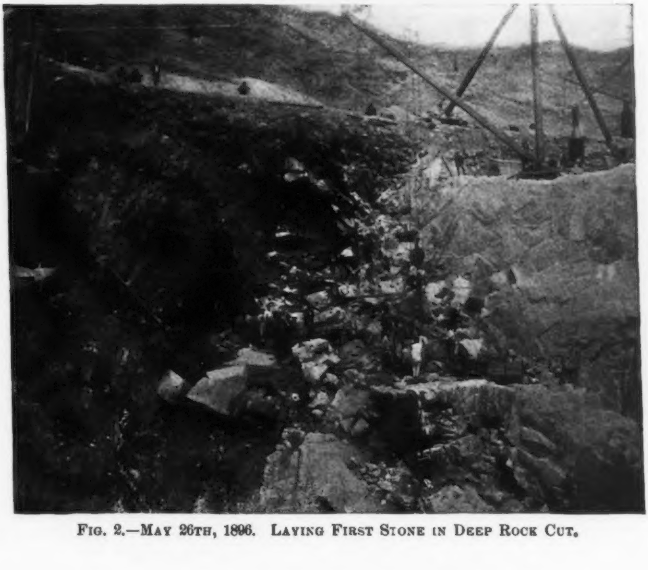


FIG. 2.—MAY 26TH, 1896. LAYING FIRST STONE IN DEEP ROCK CUT.

the seams separated the rock into heavy solid masses which they bounded on all sides. The bottom reached was solid, compact and tight. At the date of writing, a narrow strip of this bottom, close to the river wall foundation, remains to be excavated. This was left, when the great bulk of the work was completed, as, at that time, it furnished the foundation for a trestle then in use. This remaining strip is about 20 ft. wide, and, with the section of the overflow bottom now under process of work, completes all that is incomplete in the foundation excavation of the main dam.

Under and beyond the river-bed, the character of the rock changes entirely, being composed wholly of limestone. The two rocks were separated by a well-defined, nearly vertical layer of shale, black in color, especially on the up-stream side, friable on the surface, but becoming harder a few feet below, particularly on the down-stream half of the foundation. The welding of the two main rocks, the gneiss and the limestone, with the shale, appeared to be quite complete at the depth of excavation finally reached. The surface of the limestone, from the point of junction toward the south, was nearly level for a distance of about 400 ft., until it reached well into the other side of the valley, where it rose gradually with the south slope. The limestone varied greatly in character throughout the extent uncovered. In places it was of sufficient compactness and water-tightness to answer for the foundations of the structure. In other places the general character was diversified by belts of varying width which were either full of eroded seams, through which water was found to flow freely when excavation was in progress, or masses of stone broken up by seams running in all directions, which were filled with mud. In addition, there were other well-defined belts, and all followed the general dip and strike of the rock, which, in the case of the dip, was nearly vertical, and of the strike, at right angles to the line of the dam, following the valley. These last belts were of partly disintegrated, finely granulated limestone; were very well defined and at the surface were easily removable with the pick; and grew harder and more compact with increased depth of excavation. These fissured, eroded and granular belts seem to form three distinct classes into which the bad features of this limestone bottom may be separated.

The different fissures developed many erosions in certain cases and were found at various points through the limestone stretch of the

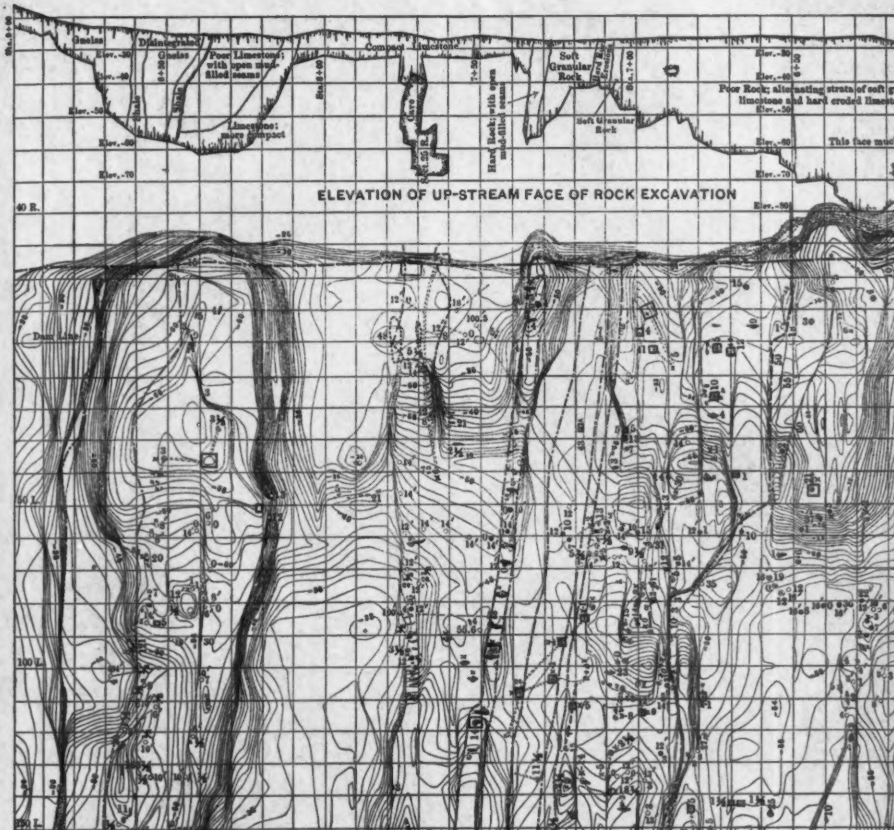
foundation, being larger and closer together as the junction with the gneiss was approached. These fissures, while well defined, were of varying widths, developing a line of erosions generally through very hard limestone.

As an illustration of the eroded seam, one case developed into a cave, the location and existence of which were noted by tracing a narrow, horizontal seam in the rock near the surface, at about Station 7 + 70, 50 L., along the strike of the rock. This seam was in fairly solid rock, and clear water flowed from it. As the excavation along the line of this flow toward the up-stream side of the dam progressed, there was found a sharp downward dip, and the flowing stream soon required for its management a subsidary pump. The seam enlarged into an erosion filled with sand, which, as it was followed, developed into a cave about 7 ft. x 9 ft. in section. This led under a heavy mass of solid rock to and beyond the up-stream line of the dam foundation. Connected with it were found, on the sides and in the roof, other erosions which were traceable nearly to the surface of the rock within the limits of the dam foundation, and which, on the up-stream side, outside face limits, in one case penetrated to the rock surface, where it showed as a narrow and somewhat prolonged fissure.*

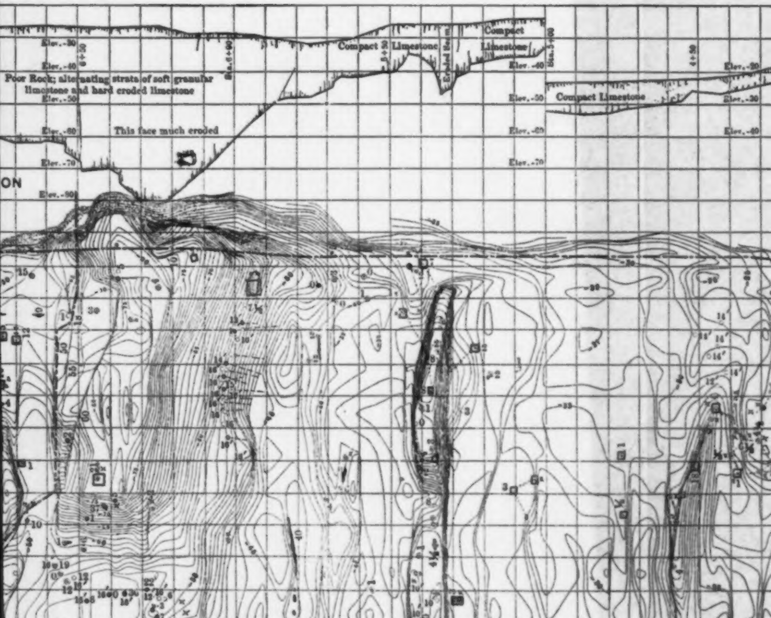
While all the eroded fissures showed flows of water of varying degrees, several such were found which developed into strong springs, of which special care had to be taken. One, in particular, was found as the excavation in the rock deepened, limited and defined to an erosion in solid rock 6 or 7 ft. in diameter at about Station 6, near the up-stream side. The flow here was continuous and heavy, more than filling the 10-in. pipe which was at first placed to receive it, and afterward, as the spring hole was welled up in the foundation masonry, rising with this masonry and in the pipes which were at the same time placed in connection with the well, to a height of 90 ft. above its source before it was found advisable and expedient to attempt to fill it up and block it off. A particular and detailed account of all the operations connected with this spring will be found in the particular description of the treatment of the rock bottom.

As to the granular belts referred to, the excavation in them was carried down until the surface exposed was very compact. These surfaces were afterward tested for bearing power by means of an arrange-

* A detailed description of this cave will be found further on.

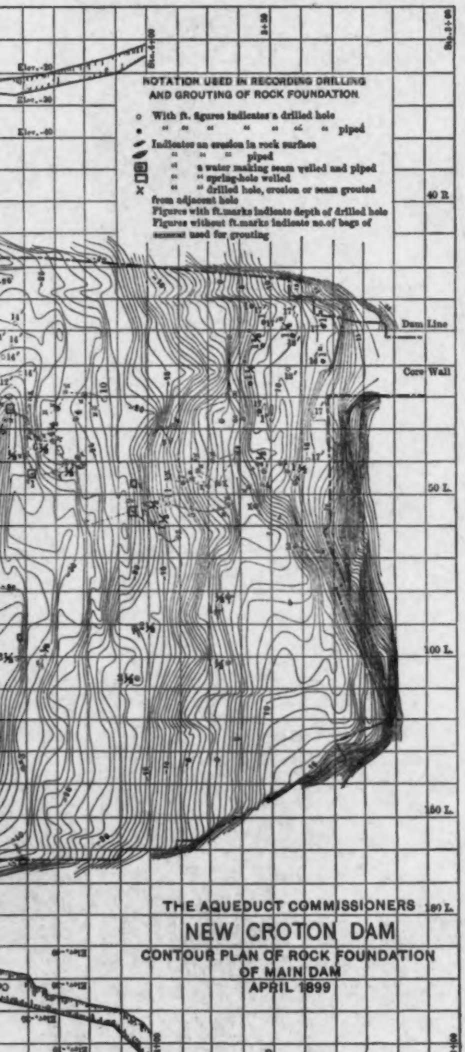


ELEVATION OF UP-STREAM FACE OF ROCK EXCAVATION
Gneiss
Disintegrated Gneiss
Poor Limestone; with open mud-filled seams
Limestone: more compact
Compact Limestone
Soft Granular Rock
Hard Rock; with open mud-filled seams
Soft Granular Rock
Poor Rock; alternating strata of soft granular limestone and hard eroded limestone
This face much
Dam Line
40 R.
50 L.
100 L.
180 L.
ELEVATION OF DOWN-STREAM FACE OF ROCK EXCAVATION



Compact
Compact Limestone
Limestone
Poor Rock; alternating strata of soft granular limestone and hard eroded limestone
This face much eroded
Compact Limestone
ON

PLATE XXXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 875.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.



ment especially designed for that purpose and shown in Fig. 10. Further allusions to these belts will also be found later.

In limiting the extent of the excavation vertically, the end aimed at was to reach rock sufficiently free from seams, and solid enough to afford all the bearing strength necessary to sustain the superimposed masonry and resulting pressure. The result involved a very large amount of deep rock excavation; the depth in one place being 50 ft. before satisfactory compact rock was found. It is not assumed that there may not be some tendency to upward pressure through some of the fissures which remained after the excavation was completed, but, as will be described later, every effort was made and every precaution taken to fill them, and it may be conceded that should upward pressure occur in some cases it must be reduced to the very small area presented by the mouth of the fissure in question to the bottom layer of the dam masonry, this area forming a very small proportion of the greater area against which upward pressure might be expected.

As to the possibility of percolation under the dam, that question would be more important if the rock bottom were exposed to the direct contact of the water in the reservoir, but it must be borne in mind that from the lowest point of the foundation of the main dam to the top of the back-filling above, there will be a compact filling of about 150 ft., in this particular case, which, while extreme, is not different, excepting in the great depth, from the condition which will obtain along the whole length of the masonry dam.

This question of possible percolation will be further considered in connection with the chapter on "Pumping."

It is an important and peculiar fact that, throughout the rock excavation of the whole foundation, in no case did the numerous test holes, drilled in the vicinity of seams and erosions, strike any openings in seams or rock which were not easily traceable by some continuous natural passage to the surface of the rock under preparation for the foundation. In other words, it may be fairly claimed that the existence of all open seams lying within 12 to 16 ft. of the dam in the various bad sections was traceable from natural indications at the surface. It is, therefore, to be assumed that all such seams were found and properly noted. A reference to the contour plan, Plate XXXVIII, will show that the variations and character of the limestone, and the necessary excavation, were much greater nearer its junction with the

gneiss than at the south end, where, with the exception of a few eroded seams, the rock is uniformly hard and compact, and required but comparatively little excavation at the surface. It would seem that at some time the disturbance of the limestone formation must have been considerable; the greater part of it occurring near the point of junction. From developments indicated by a comparatively small amount of excavation in this part of the limestone foundation, and the fact that the general character of this bottom was naturally considered an important matter, it was deemed advisable, during the excavation of the first section of the bad rock, which lay at Station 8 + 50, to consult a specialist as to the general condition in which limestone ledges might be expected to be found under the prevailing conditions, and Professor Kemp, of Columbia College, was consulted, and his attention was called particularly to the question of the probable existence of caves and similar openings under the general rock surface. The following is his report, which is introduced here as being of interest under the circumstances :

"New York, May 14th, 1896.

"Mr. A. Fteley,

"*Chief Engineer, Aqueduct Commission.*

"My Dear Sir,—In reply to your letter of the 12th, requesting me to report also upon the probable condition of the limestone under the site of the dam, I append the following to my report of two days ago.

"The limestone is undoubtedly more or less fissured precisely as is the gneiss and as is to be expected in regions where the rocks have been upturned to a vertical position.

"Such small cracks cannot of course be avoided and, I understand, are not matters of serious concern. They are the ones that now show in the walls of the pit and that let in the water in all probability from the overflowing water-soaked sands and gravels.

"As to the presence of large caves, several feet across or more, and of great length, I am of the opinion that their existence is improbable, and so improbable as not to give occasion for special treatment. I think the points *For* and *Against* them may be stated as follows:

"For.

"1. The rock is limestone, and caves are practically limited to limestone; other soluble rocks being rare with us.

"2. Mr. Value has been impressed with the fact that the water trickling into the sump has diminished as the pit has gone deeper. The inference has been suggested from this observation that the water has run away into some underground cavity. (See further, under 5, below.)

"A third point is stated and discussed under 2, below.

PLATE XXXIX.
TRANS. AM. SCC. CIV. ENGRS.
VOL. XLIII, No. 875.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.

FIG. 1.—MAY 28TH, 1896. MAIN DAM EXCAVATION. LOOKING SOUTHWEST.

FIG. 2.—MAY 19TH, 1896. MAIN DAM EXCAVATION. DEEP ROCK CUT, LOOKING WEST.

"Against.

"1. Caves only form above the level of the ground-water or well-water, because only freshly fallen rain is sufficiently charged with carbonic acid to be a strong enough solvent to be serious, and because only water in this situation flows rapidly enough to produce profound effects. The ground-water stands too still, and too soon becomes saturated with lime, to be effective. The present position of the rocks is below the zone at which caves could form, and it is practically assured that none have formed since they assumed this position.

"2. If any have formed, they must have done so when the rocks stood at a higher position and above the ground-water. We all believe that this whole region was much elevated during the Glacial period, and it cannot be denied that conditions may have been favorable at that time. Some superficial decay apparently took place, as shown by the sandy streaks in the limestone, but after this time a strong stream must have flowed over these rocks to have availed to deposit the heavy burden of sands and gravel that rest upon them, and if any such cavity existed near the surface the probability is strong that it has been packed full of sand.

"3. The rocks stand vertically, and all underground drainage or circulation must tend to follow their bedding planes much more than to cross them. We would infer from this that any cavity would be long and narrow and not an easy thing to locate with a drill.

"4. No hollow sound, so far as I know, has been noted in the work in the pit, when picks, drills or the descending boxes from the cables have struck the bed-rock.

"5. In case the water has diminished, as observed by Mr. Value, I think it is due to the partial exhaustion of the neighboring gravels, for the weather has been dry and rainless for a long period, rather than to any cavities under the bottom of the pit. Such assumed cavities, being 50 to 100 ft. below the level of the Hudson River, and having stood for an indefinitely long time under wet gravels, must have been long since filled with water.

"6. All the experience, so far as I know, that has been gained in quarries in these limestone belts in New York and the neighboring parts of New England, has shown caves to be extremely rare. An assistant of mine has recently had occasion to visit every one of them, and he only met one small cave, which was at Hastings. Of course there may be others, and I am aware of the existence of a large cave near the Twin Lakes in the northwest corner of Connecticut, but, considering the abundance of the limestone areas, they are certainly rare.

"For these reasons I regard the probability of their existence under the site of the dam as remote.

"Very respectfully yours,

"(Signed) J. F. Kemp."

In order to explore this limestone bottom more completely, it was found advisable to drill a few test holes of considerable depth. These were accordingly undertaken at certain points in the bottom where it was nearly ready for the masonry, and their location, direction and depth, are shown on the contour plan.

An extended account and description of the excavated rock bottom, for the main dam foundation, referring particularly to the limestone bottom and to the various changes and characteristics shown by this rock, can be found in a report made by the author to the Chief Engineer of the Aqueduct Commission, in which, for the purpose of a record, all the facts are noted in considerable detail. Constant reference is had to Plate XXXVIII, which is a plan in contour of the finished rock bottom between Station 3 + 20 and Station 9, and which shows contour elevations at intervals of 1 ft. On it are shown also, by means of cross lines, the limits of the different belts in the rock, and the heavy dotted line shows the neat lines of the dam foundation masonry.

The shale seam, separating the gneiss and limestone, lies at Station 8 + 80 ±, varied in width at different points, but grew narrower and more solid as the depth of excavation increased. Fig. 2, Plate XXXVII, shows the shale in the face of the cut and in the trench bottom on the left, where, however, the excavation shown is unfinished. The view is taken looking up stream.

The next of the series of belts into which the foundation may be divided, and which are in a measure indicated by the profiles of the finished bottom shown in Plate XXXVIII, and in Figs. 4, 5 and 6, extends from Station 8 + 20 to Station 8 + 70. Its character, when the excavation was about completed, is shown in Fig. 2, Plate XXXVII, and Figs. 1 and 2, Plate XXXIX. The contour plan, Plate XXXVIII, shows the number and depth of the search holes drilled in preparing the bottom for masonry. In this case they followed principally the lines of the erosions in the hard rock bottom.

Next beyond lies a section, showing also in Fig. 1, Plate XXXIX, between Stations 7 + 80 and 8 + 20, and forming a solid ridge of hard, compact limestone, requiring but little excavation, comparatively.

A well-defined narrow seam along Station 7 + 70 is illustrated by Fig. 1, Plate XL, showing its down-stream end. Its up-stream end

Fig. 1.—September 1st, 1896. Main Dam Excavation, Looking West.

Fig. 2.—September 12th, 1896. Cave at Station 7 + 70, 10 L.

developed upon excavation into the cave previously referred to. Fig. 2, Plate XL, shows the cave opening at Station 7 + 70, 25 L. The pump and suction hose in use are also shown. This suction hose and another line, of the same size, were built in the masonry when the tunnel was filled up, in order that the necessary drainage from a sump hole, placed outside of the upper line of the dam in the lowest point of the tunnel reached, might be maintained. The length of the cave excavated and filled as tunnel was about 30 ft. The floor is of very hard and solid rock; holes traced 16 ft. deep found no openings below. The masses of rock on the top and sides are all solid, showing few or no open seams, except that the seam between the cave floor and the right side wall may have had an open connection with the low point in the excavation at Station 7 + 35, 15 R., where, in grouting, later, there were some indications of an open passage between the points in question. As this was indicated by the pump from the sump at Station 7 + 62, 25 R., throwing out grout which was being pumped in at the point noted, it is not at all certain that the line of communication between the two did not lie mostly outside the dam foundation.

To facilitate the work of filling up the cave, a small shaft was sunk at Station 7 + 73, 23 R., to strike one of the subsidiary caves found on the upper line of the dam. Two other and smaller eroded chambers, leading into the roof of the main cave, were also found. These spaces were all filled, within the lines of the dam, with rubble masonry, or, in the case of the two small caves shown in plan on the contour plan at Station 7 + 70, and Station 7 + 78, with well-packed small stone, placed from above through openings made for the purpose in the overhead rock, and then filled with grout.

The sections and sketches opposite Station 7 + 70, Fig. 7, show the location and extent of these various caves, and it may be noted that in filling up the main cave and the branch cave, *i. e.*, entering above the main cave roof from above the up-stream line of the foundation masonry, care was taken to build this masonry filling 4 ft. beyond the limiting line. The large cave beyond the line was found to be about half full of sand and gravel when it was reached from the tunnel and the small shaft sunk in its roof. It is evident that originally the space had been solidly filled, but that the cleaning of the tunnel, and the pumping of the heavy water flow had caused the partial emptying of the filled space beyond. It was refilled later, after the masonry

filling had been built, by washing gravel down from the slope outside through openings made in the surface of the rock for that purpose.

Between Stations 7 + 30 and 7 + 60 is shown a narrow well-defined seam of hard rock with many erosions connected and extending to the deeper holes excavated at the ends. Fig. 1, Plate XL, shows on the left the deep excavation on the down-stream end. Beyond this seam lies a compact seam of friable limestone about 10 ft. wide. It is shown in Fig. 1, Plate XL, on the extreme left, where the ladder is resting against it, when the excavation was completed. It was tested for bearing strength by an apparatus shown in Fig. 10. This apparatus consists of a cylinder to be loaded with shot necessary to produce the required pressure upon its bearing point, a circle ½ in. in diameter. This was applied carefully and repeatedly to the surface in question at different points, and the results indicated that the bearing power of the surface was ample up to the limit of the test, which was 250 lbs. to the square inch.

A second but much narrower section of the soft granular rock just mentioned lies a short distance beyond. This shows on Plate XL, as from 3 to 5 ft. in width, and beyond this is a wide stretch of bottom shown on the plan as being composed of alternate strata of "soft and granular limestone and hard eroded limestone." In this the most extensive and deepest excavation occurred. Figs. 1 and 2, Plate XLI, which are views looking up stream, illustrate the character of the excavation. Fig. 2, Plate XLI, shows the deepest point reached, *i. e.*, Elevation—80.4 below datum. Figs. 1 and 2, Plate XLII, show the character of the rock in the same vicinity more in detail, and the location of some of the erosions through which excavation was made. In Fig. 2, Plate XLI, is shown the spring hole at Station 5 + 93, with the iron pipe in use to convey the flow.

The sloping bed of rock shown in Fig. 2, Plate XLI, extends quite across the dam; and beyond it, to Station 5 + 40, lies a hard bottom, reasonably free from seams and erosions, and calling for but little excavation. Between Stations 5 + 30 and 5 + 40 occurs a deep, well-defined seam, or series of erosions, shown at the upper end in Fig. 1, Plate XLIII. This photograph shows also the general character of the rock bottom on both sides of this seam, and Fig. 2, Plate XLIII, and Fig. 1, Plate XLIV, show it still further to the south, and to the junction of the main dam with the core-wall. This part of the bottom

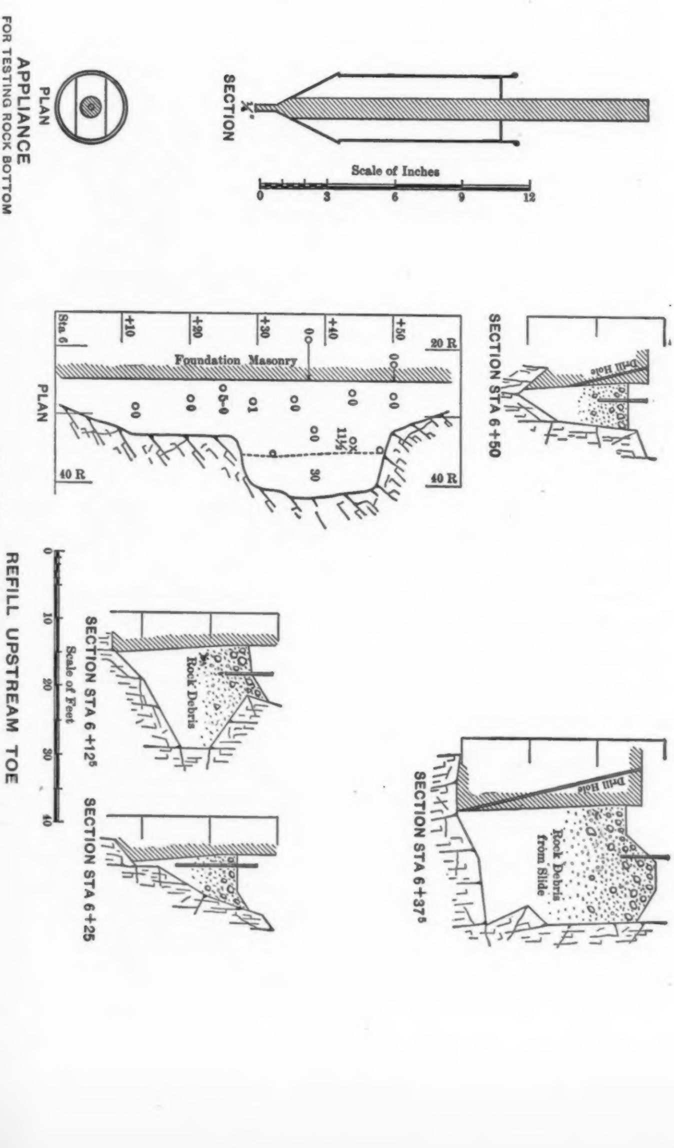

SECTION
PLAN
APPLIANCE
FOR TESTING ROCK BOTTOM
Scale of Inches
0
3
6
9
12
SECTION STA 6+50
Drill Hole
Foundation Masonry
Sta 6
+10
+20
+30
+40
+50
20 R
40 R
40 R
PLAN
SECTION STA 6+37⁵
Rock Debris from Slide
Drill Hole
SECTION STA 6+12⁵
Rock Debris
SECTION STA 6+25
Scale of Feet
0
10
20
30
40
REFILL UPSTREAM TOE

Fig. 10.

calls for little comment, although a seam showing at Station 4 + 60, and extending partly across the foundation, is shown nearly ready for masonry in Fig. 1, Plate XLIV.

In addition to the search holes, which were numerous, and were from 12 to 14 ft. in depth, it was thought advisable to drill a few test holes of considerable depth. They were accordingly undertaken at certain points in the bottom where it was nearly ready for masonry, and their location, direction and depth are shown on the contour plan. The first, or No. 1, was located at Station 8 + 68, 103 L. Length drilled, 48.4 ft.; direction and dip indicated by the black arrow ending at 8 + 52, 100 L. Hole No. 2 was located at Station 7 + 76, 84 L. Length drilled, 100.6 ft.; direction and depth indicated by arrow ending at Station 7 + 16, 86 L. Hole No. 3 located at Station 7 + 52, 5 R. Length drilled, 100.6 ft.; direction and dip indicated by arrow ending at Station 7 + 14, 50 L. Hole No. 4 is located at Station 7 + 50.5, 75.8 L., and was drilled vertically 55.6 ft. deep.

The three inclined holes were drilled in May, June and July, 1897, while the sump hole at Station 8 + 50, 10 L., the bottom of which was at Elevation —67, was in use. They were inclined in order to cross the vertical seams, and to better the chance of finding large erosions or caves. The results did not seem to indicate anything more extensive in the way of erosions than had already been found by the excavation for the first bottom formed between Stations 8 + 50 and 8 + 70, and it was some time later in the season that the excavation in its regular progress developed the location of the large cave to which attention has been called.

Grouting and General Treatment of the Rock Foundation.

Upon the completion of the rock excavation of any particular section of the bottom, which work included in many places a prolonged and tedious barring out and cleaning up of shaky or loose pieces of rock, the bottom was washed down and thoroughly cleaned by streams of water under a heavy head; and operations were then begun to clean out all erosions and open seams showing at the surface, and to trace them out as thoroughly and as far as possible by drilling numerous holes of varying depths in their vicinity.

All erosions and open seams were, as a rule, filled with Portland

FIG. 1.—NOVEMBER 20TH, 1896. MAIN DAM EXCAVATION. ROCK FACE, UP-STREAM SIDE.

FIG. 2.—FEBRUARY 26TH, 1897. MAIN DAM ROCK EXCAVATION AND MASONRY.

cement grout mixed with fine sharp sand of 1 to 1 or 2 to 1 mixture according as the grout was pumped or poured. In the case of the "cave," rubble masonry in mortar was used as filling in the large opening, while in some smaller erosions the spaces were thoroughly packed with small stônes before the grout was poured in, and, in one or two exceptional cases, American cement was used for the grouting.

The drilling of holes in the rock bottom for grouting and searching purposes was begun as soon as the first section of bottom was excavated. Air drills were used, and were fed from the pipes used for drills at work on the general excavation. In all, about 1 700 lin. ft. of holes were drilled. They were about 2½ ins. in diameter at the rock surface, decreasing somewhat as the depth increased. These holes were of all depths up to about 16 ft., according to circumstances. Whenever it was found impracticable or inadvisable to pour or pump grout into holes or erosions before the adjacent masonry work was started, vertical pipes, generally 2 ins. in diameter, were placed in them and were then built around with the masonry up to such height as was necessary. In case holes or erosions showed a flow of water, such openings were also provided with pipes placed at proper inclinations to lead to some drain center or sump hole near by. In most cases the erosions, and in a majority of cases the drilled holes, were piped, as it was found more convenient and practicable to pump grout under heavy pressure into openings thus prepared and sealed and covered with masonry carried up to some convenient height. Old steam-piping was generally used for this purpose, and, while the diameter was generally 2 ins., other and larger sizes were sometimes provided, and, in the bottom, hard tile piping was at times used to carry water flows. In the case of the heavy spring at Station 5 + 95 large-sized, galvanized, riveted pipe was used to connect with the spring as the foundation masonry was built up, and at the cave at Station 7 + 70 it was found necessary to build into the filling masonry two 10-in. galvanized-iron suction pipes which were afterward filled by pumping grout.

Whenever grout was pumped a No. 2 Douglass deck pump was used. The grout was mixed by hand in boxes made for the purpose. The suction and delivery hose were each 3 ins. interior diameter, coupling to 2-in. hose at the pumps and also to a 2-in. nozzle, at the outer end of the discharge hose, made of a short piece of steam pipe. When

in use, this nozzle was either set well into the pipe leading to the channel to be grouted and carefully packed with waste and old bagging, or, in some cases, was coupled to the grout pipes by the use of screw threads and couplings, cut and furnished for that purpose. There were from four to six men at the pump handles, according to the resistance experienced in forcing the grout, and the pressure developed was sufficient in cases to burst the hose while still forcing appreciable quantities of grout to flow. As a preliminary to grouting any hole or section, care was always taken to flush the pipe and passage very thoroughly with water under a heavy head; a system of pipes leading from the Old Aqueduct, which was at an elevation of from 200 to 250 ft. above the main dam foundation, furnishing all the facilities for this purpose.

The contour plan, Plate XXXVIII, shows the position and extent of the various fissure erosions, cave and springs treated, together with the location and depths of all holes drilled in the process of tracing out; and also figures showing the number of bags of cement used in grouting at various places.

The first section grouted was between Stations 8 + 20 and 8 + 70. The search holes drilled had in many cases established a connection with the lines of erosions showing in the bottoms, and most of the holes were filled by the flow from holes adjoining. In this section 701 bags of Portland cement (175 bbls.) were used, mixed with sand (2 to 1). All erosions of any size were filled with small stones before being treated with grout.

The next section treated was along line 7 + 70, and included the cave which, after being cleared of gravel and having control of the water flow, gained by means of a 10-in. double Worthington pump (1 500 000 galls. per 24 hours), was filled with rubble masonry laid in Portland cement mortar. The sump-hole was established outside the line of the dam at about 27 R., and its location has been shown at the ends of the two suction pipes which, as stated, were left in and built around solid with masonry. The larger pipe was the one used in connection with the 10-in. pump. The second pipe was placed about 2 ft. higher than the other and was provided in order that drainage might be maintained in case the lower pipe should clog with sand or gravel during the filling up of the cave. A third pipe 3 ins. in diameter, reaching partly through the cave, was laid on the bottom below the

PLATE XLII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 875.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.

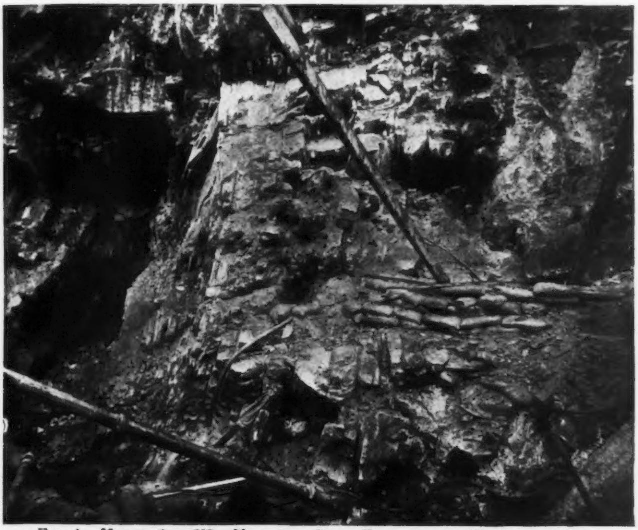


FIG. 1.—MARCH 2D, 1897. MAIN DAM ROCK EXCAVATION. UP-STREAM FACE.

FIG. 2.—FEBRUARY 17TH, 1897. ROCK BOTTOM AND EROSIONS.

lower suction pipe and reached to the sump-hole. It was used to take slight flows from the walls and floor to the sump, and was also built in when the cave was filled. During this process the lower suction became clogged and the pump was connected temporarily with the upper pipe, while a stream of compressed air was blown through the 3-in. pipe. This freed the outer end of the lower suction which was again put in use, and no further trouble was had with it.

Fig. 7 shows cross-sections of the cave at various points. The section at 00 D. L. shows on the left the connection with the long oval-shaped pocket shown on the plan at Station 7 + 78. It also shows on the right a connection with a smaller pocket at Station 7 + 70. These pockets were comparatively deep, and plainly showed erosion between the solid stratum forming the cave roof and the somewhat softer stone on each side. The section at 125 R. shows the average cave section inside the masonry lines. At 25 R., just outside the up-stream line, the cave abruptly enlarges, reaching nearly to the rock surface, while at 37½ R., the section is somewhat smaller apparently, although it was not free enough of gravel and sand to show that clearly.

As the cave was cleared out it was heavily timbered in the roof for the protection of the workmen from the possible fall of detached pieces of rock, and, when 27 R. was reached, a timber bulkhead about 4 ft. high was built to retain the gravel slope lying in the fissure beyond. It was also of use in forming the outer wall of the sump-hole which was located on the extreme right of the cave where the roof was low. A shaft 4 to 6 ft. square and about 6 ft. deep, between Stations 7 + 69 and 7 + 75, was sunk to reach the roof of the enlarged cave section which, at this point, ran back about 10 ft. from the outer neat line of the dam, re-entering for that distance over the roof of the cave proper and near the rock surface. The masonry filling began close to the bulkhead line, 27 R., and was carried up on that line vertically to the surface through the 4 x 6-ft. shaft which was excavated for that purpose. As this filling progressed in the cave, working toward the center of the dam, the timber was gradually removed with the exception of two 8 x 12-in. range timbers which extended throughout its whole length and which were built in. A small shaft was sunk into the roof of the oval-shaped pocket on the left, at Station 7 + 78, and when the cave masonry below had been built to the general roof level this pocket was filled with small stones and then grouted, taking

forty-eight bags of cement (2 to 1 mixture). The smaller pocket on the right was packed with stones from below and grouted through an inclined drilled hole 12 ft. deep, taking eight bags of cement (2 to 1 mixture). Other inclined holes were drilled in this vicinity—12 to 18 ft. in depth—in a search for further cavities.

To the pump ends of the suction pipes as they were built in, reducers and 2-in. iron pipes were finally attached, and the water from the sump-hole outside was allowed for a long time to flow through and was conducted to a temporary sump-hole near the center of the work, while the masonry was gradually built up. As these pipes were raised higher, this flow finally stopped, as the back-filling on the up-stream side then in place was not sufficient to prevent the flow from finding an outlet in the up-stream sump to the south. These suction pipes were grouted when the masonry and connecting pipes had been raised about 40 ft., 72 bags of Rosendale cement (1 to 1 mixture) were poured into the lower and longer pipe, filling it, then 42 bags of Rosendale cement (1 to 1) were partly poured and partly pumped into the upper and shorter pipe. The pumped material was forced up through the back-filling on the up-stream side and this caused a temporary stopping of the experiment. Some time later (about a year), 11 bags more were pumped in, and the hole was blocked, no sign of this grout showing this time in the up-stream back filling, which, in the meantime, had been carried up much higher. In the narrow, eroded seam lying along the line from Station 7 + 32, up stream, to Station 7 + 58 down stream, 300 bags (75 bbls.) of cement were used, the grouting showing at least connections between adjacent erosions and search holes as the seam was pumped full.

Beyond, between Stations 6 + 80 and 7 + 10, and partly including a bottom which was hard and solid, but full of open seams and erosions, and distinguished by some solid masses which rose above the general surface, a great many pipes were used, and a large quantity of grout was pumped in.

The next bottom section, covering the lowest point reached for a foundation, was drilled and treated as usual, but the extreme low bottom on the up-stream side took but little grout except along the line of seams from 20 R. to 30 L. near Station 6 + 50.

On the down-stream side will be noted some lines of erosions into which considerable grout was pumped.

PLATE XLIII.
TRANS. AM. SOC. CIV. ENGRS.
VCL. XLIII, No. 875.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.

FIG. 1.—JUNE 27TH, 1897. ROCK BOTTOM, SHOWING ERODED SEAM.

FIG. 2.—AUGUST 12TH, 1897. ROCK EXCAVATION, LOOKING SOUTH FROM STATION 6 + 00, 60 L.

But little grout was used beyond this bottom until the eroded seam between Stations 5 + 30 and 5 + 40 was reached, as the spring hole at Station 5 + 93 was not treated by grouting.

The erosions along the 5 + 40 line were drained at a sump-hole at 39 L. during the excavation, while the bottom masonry was being laid and the drilled holes and erosions were being piped. A well was therefore gradually built up at this point, reaching a depth or height of about 20 ft. before the grouting work was started. The holes in the seam between the well and the up-stream side were grouted by pumping before the well had reached this height, as there was no connection between them, the water in the well coming wholly from the other direction. When the well was ready the drainage pump was taken out and the drainage was maintained by a pump attached to the 5 in. pipe shown at 165 L. This pipe was just outside the down-stream toe line of the dam, and had been placed and used for a drainage well while the rock excavation in its vicinity was being made.

The main well hole, as it was built up, in places had its down-stream face built of stones laid dry, in order that seams in the adjoining rock might not be shut off from the grout later, as well as to allow free passage of the water to the suction pipes. A 2-in. pipe was also built into this well, reaching to its lowest point and connecting there with seams in the rock.

The well was filled with 80 bags of Portland cement (1 to 1 mixture) poured in, and it was evident from the water which was forced from pipes nearby, notably at 54 L., that the grout was reaching the seams and passages in that direction. As the grout was poured the well was gradually filled with small stones collected for that purpose. After no more grout could be poured 4 bags of cement (1 to 1 mixture) were pumped into the pipe placed in its corner. The grout pump was then tried in each pipe in turn working toward the down-stream side. The grout was forced gradually into the 5-in. pipe, the pump at which was stopped when the grout trace became marked. This pipe was filled, so that the water flow ceased through it, by pumping at the spring holes near by at 150 and 158 L. Later, after the pump was disconnected, it was completely filled by pouring 5 bags of cement (1 to 1 mixture) into it at the top. By this time, the water which had flowed along this seam was blocked off entirely and had forced its way up to

the surface of the down-stream gravel slope at an elevation considerably above the top of the 5-in. pipe.

Beyond this seam, little grouting was found necessary until Station 4 + 60 was reached, where the seam developed in the course of the rock excavation showed some traces of erosion on the solid vertical face left in on the north side. The nearly horizontal open seam reaching under this vertical face showed a few springs, and at two places, 43 and 126 L., 18 and 30 bags of cement (1 to 1 mixture), respectively, were pumped in, and smaller quantities at other places. The spring at 43 L. was filled and "X"d at three points between 25 and 31 L. The grouting nearer the down-stream side gradually drove the water outside the masonry limits. Six holes, 12 to 14 ft. in depth, were drilled on the line of this seam near the up-stream side, but further traces of it were not found. A number of pipes were placed between 18 and 45 L., Station 4 + 10 to Station 4 + 40, above and along very narrow but somewhat open seams in masses of solid white rock, and, as the bottom masonry was laid, these pipes were connected by covering the seams with small spawls laid dry. The pumping afterward done indicated free flowing between the pipes, and a considerable portion of the grout must have been used to fill channels thus provided.

The same remarks apply to the piped seams from 40 to 60 L., Station 3 + 50 to Station 4 + 10, where the open seams were in most cases so fine that they could have taken but little of the grout pumped. A wider seam at Station 3 + 75, 0 to 30 L., was piped and took an appreciable quantity, as is shown on the plan. There were no signs of erosion there.

The drilled holes, 16 to 18 ft. deep, between Stations 3 + 30 and 3 + 60, practically took no grout. Neat cement was pumped into them, and the amount taken was only about enough to fill the holes.

They were drilled, as the rock, though compact and fairly solid, was full of short heads and tight seams, and it was thought best to make sure that the seams were no looser below.

The Spring at Station 5 + 93.

It was at first proposed to grout this spring, as well as the others, but circumstances led finally to a different course of treatment in this case.

The flow from this spring was heavy. When first uncovered it was curbed with a bag dam and piped as shown in Fig. 2, Plate XLI. The

pipe was 8 ins. in diameter, and the flow was sufficient to back up against the pipe at the entrance. The inclination of the pipe, however, helped the flow, and at its lower end the pipe was about half full. It was unfortunate that no gauging of this flow was ever made, but, with many other springs and flows about, it was overlooked.

The flow was carried through this pipe for some months until the masonry work had reached the spring level, when it was taken by a 12-in. iron pipe laid in the masonry to the sump-hole by this time established near by, outside the up-stream face which had been carried up to about 30 ft. above the lowest point of the rock bottom. Later, another 12-in. pipe was laid from a slightly higher elevation through the masonry outside the face of the dam. This pipe was continued with a 90° elbow, and, after plugging the lower pipe, short vertical lengths of pipe were added to it by which the point of discharge was gradually raised as the masonry forming the well above the spring was carried up ahead of the discharge pipe. This arrangement also allowed the back-filling against the up-stream face to be carried on conveniently without impeding the flow of the spring or filling it with earthy material. The head of the spring was reached at about Elevation + 20, when the flow, which had been gradually diminishing, ceased. This was about 83 ft. above the rock bottom of the spring hole and 74 ft. above the outlet pipe which by this time had served its purpose for 15 months while the masonry was building. The section in Fig. 9 shows the above-described features, as well as the partial location and trend of the open fissure leading from the spring hole. This fissure showed at the well hole a considerable section; its direction was down stream with a downward horizontal dip. Viewed from above the well hole, the bottom was full of well-washed gravel stones of comparatively small size. As the passage was always full of water flowing swiftly, it could not well be explored, but the line of drilled holes, shown on the contour plan, from 7 to 17 ft. in depth, was used to locate its position and direction as far as practicable. Five of these holes reached the fissure or connecting seams and were piped with 2-in. pipes until the head of the spring was reached. The additional sections shown in Fig. 9 are from the results of these borings. When the well had been built to Elevation — 2, it was decided to seal it, as to carry it higher involved unnecessary complications with the masonry work. It was 4 ft. square. A 3-in. iron pipe

was placed in one corner and reached nearly to the bottom. A large flat stone was lowered, and by tag and guide ropes so placed as to partly cover and shield the 12-in. pipe opening. Its position was directed by sounding with plummet and wire line, and two trials only were necessary to place the stone properly. The well was then filled up to Elevation — 47, about 8 ft. above the outlet pipe, with clean spawls, the larger sizes being placed on the bottom and gradually diminishing to the size of concrete material at the top. These stones were lowered to place in a box built to fit the well, and with a bottom which could be tripped open. The same box was used to continue the filling with concrete, Portland cement in the proportion of 4 gravel, 2 sand and 1 cement being used. The dump box had a capacity of 18 cu. ft. The first batch of concrete was mixed dry and was placed in the box on a sheet of canvas which covered the bottom and sides of the interior, and formed a tight bottom for the concrete mixture when it was dumped. The dumping of the concrete was continued diligently until it had risen about 13 ft. This work had not in any way disturbed the flow through the outlet pipe, which showed no discoloration due to cement or gravel, and examination of the 3-in. pipe in the well corner indicated that the sealing was complete, as the displaced water in the well was overflowing at the top, while the water in the 3-in. pipe remained stationary. This was further shown the next day when the well was bailed out nearly dry to facilitate its further filling with concrete. When filled the masonry work above and around the well was resumed, but the 3-in. pipe was carried up with the others until the head of the spring was reached.

One of the four 2-in. pipes furthest away from the well hole took water freely. They were about 90 ft. long. The water from the spring rose in the line of pipes to Elevation 20 ±, about 10 ft. below their tops. It was a question whether grout could be poured successfully through so much water, and in the first pipe tried, 17 L., the grout clogged half-way down the pipe, owing, apparently, to some roughness due to carelessness in joining the sections of the pipe. This was washed out by a flow of water pumped through a ½-in. pipe, and the pipe was cleared.

It was then suggested that an effort be made to fill this erosion with plastic clay by driving through the pipes. This suggestion came from the contractors, who had used clay to fill cavities under some-

what different conditions. Arrangements were made to try this method, and a small pile-driver with a 2 000 lb. hammer was set up over the 2-in. hole at 32 L.

A piece of 3-in. steam-pipe, about 6 ft. long, was used at first as a receiving cylinder, and it was connected to the 2-in. pipe which projected above the masonry about 1 ft., with a strong reducing coupling. The follower used was a $2\frac{7}{8}$-in. steel rod, of the length of the cylinder, turned to fit closely its full length. It was welded at its upper end to a lengthening rod, slightly smaller in diameter, which was fastened at its upper end to a wooden cross-head, designed to work in the guides of the pile-driver and to take the blow of the hammer.

Blue clay of good quality was used, and was made plastic with water and a thorough working and pounding into boxes 10 ins. deep and of a size to hold 10 cu. ft. These boxes were limited in dimensions simply for convenience in keeping record of the clay used. The clay was then cut into "sausages" 10 ins. long and $2\frac{3}{4}$ ins. in diameter by shovels which had been bent and sharpened for the purpose. As often as the shovel was used to make a cut it was dipped in a pail of water in order to lubricate its surface and free itself for the next cut. The sausages were passed to the pile-driver in boxes holding 50 lbs., and the amounts of clay thus determined were used for the purposes of a record, and later, were reduced to cubic yards.

The first hole to be tried was one which took water very slowly. On starting the clay driving, the machine worked very satisfactorily, but the clay drove hard and only $3\frac{1}{3}$ cu. ft. were driven in all; probably not much more than enough to fill the pipe, which was about 90 ft. long, and the fissure at its bottom. The next hole tried was at 17 L. It was also a slow water hole. Into this 332 lbs. of clay were driven, with the 3-in. cylinder, everything working hard, and an 18-in. drop of the hammer being necessary. It was then decided to change the 3-in. cylinder for a 2-in., with, of course, a correspondingly smaller piston. This piston proved to be somewhat loose in its fit, but the driving continued to be very hard and only 37 lbs. of clay in addition, were put into this hole.

The driving was then resumed at the hole at 10 L., the 3-in. cylinder being again used while a new piston was being fitted for the 2-in. cylinder. The hole took water very freely, and 200 lbs. of clay were driven easily, using a 1 ft. drop. The next 200 lbs. went harder and

required a 3-ft. drop before it was all in. A change was then made to the 2-in. cylinder and piston, but the difficulty of driving seemed to increase, although a drop of 4 ft. was given the hammer. With this, only 58 lbs. more were driven in, and then the pipe split between the coupling at the foot of the cylinder and the masonry. After repairing this break driving was resumed, with a 4-ft. drop of the hammer, for 2 hours, and very little clay was gotten in, when it was noticed that everything worked more easily and a drop of only 2 ft. was necessary. In the course of another 2 hours less drop was used and cracks began to be noticed in the surface of the masonry radiating from the pipe in use. Further effort confirmed this, and, on the next day, observations with level and transit, during a short period of driving, in which 50 lbs. were easily put into the hole, showed a distinct and appreciable rise in the masonry surface. In all, about 150 lbs. were driven in this pipe after the split had been repaired.

Leaving the upheaved masonry to be investigated later, the driving was transferred to the 3-in. pipe which had been built into the well at Station 5 + 93 as it was filled up. There was no question about a free flow through this pipe and a plumb-bob dropped in readily found bottom in the well below the known elevation of the bottom of the pipe. However, in view of the difficulty experienced in driving clay through the 2-in. pipes already tried, it was thought advisable to make a test to determine to what extent, if at all, skin friction interfered with the passage of the clay.

The 3-in. cylinder was therefore connected by a quarter turn with 97 ft. of 3-in. pipe resting on the masonry surface. At the further end another quarter turn and a 2-ft. length of pipe gave opportunity to fit a poppet valve, which was set at 34 lbs. per square inch. The pipe was then filled with water, and clay was gradually forced in from the cylinder end. It was found to require no force beyond the weight of the hammer without impact and the water was forced through the poppet valve as fast as the clay was pushed in at the outer end. Later, 28½ ft. of 2-in. pipe and 20 ft. of 1½-in. pipe were joined to the 3-in. pipe and the valve fixed at the outer end. Under these circumstances it took a drop of about 14 ins. to force clay to the extreme end and through the valve. An examination of the clay as it was forced from the ends of the various sizes of pipes showed clearly that, under even a very slight compression, the water is driven to the clay surface next

the interior surface of the pipes and acts as an efficient lubricator, the skin friction amounting to practically nothing.

Driving was then resumed at the 3-in. pipe; 3 250 lbs. of clay were forced in, only the weight of the hammer being needed on the first day.

Appended is an abstract from the log of the clay driving, following the work above noted:

"Tuesday, December 27th, 1898.—4 500 lbs. driven in 3-in. pipe; hammer only used; no impact to force clay. No evidence of clay in 8-in. pipe outside of masonry, though distinct tremor in water was noticed during driving. Water in 8-in. pipe remained at constant height, about 6 ft. from top of pipe. Elevation, 18.4.

"Wednesday, December 28th, 1898.—4 400 lbs. put in. About 8.45 A. M. water in 8-in. pipe had risen to within 20 ins. of top, and rose 5 ins. at each stroke of piston, falling back to old level after stroke. About 10 A. M. water began to flow over edge of pipe at each charge, but settled back below edge after stroke. About 12.00 M. water ceased to settle back. About 3.00 P. M. began to run in a small stream after stroke had been made. Plumbed pipe, but found no clay in it. Distance, measured from top down, 81.04 ft.

"Thursday, December 29th, 1898.—A small stream of water was flowing from 8-in. pipe when work started. 5 000 lbs. driven to-day. Plumbed 8-in. pipe as follows:

9.00 A. M.	Distance	81.04 ft.	—No clay in pipe.
2.50 P. M.	"	78.62 "	—2.42 ft. of clay in pipe.
3.50 "	"	78.08 "	—Rise of 0.54 ft. of clay with 750 lbs. put in between 2.50 and 3.50 P. M.

"At 2.50 P. M. water flowing from pipe was strongly colored with clay, which gradually cleared, and about 4.00 P. M. no trace of color could be detected in water flow.

"Friday, December 30th, 1898.—Clay driven 5 200 lbs. Plumbed 8-in. pipe as follows:

9.20 A. M.	Distance	77.28 ft.	—Rise 0.80 ft.	1 300 lbs.	clay.
11.20 "	"	76.28 "	— " 1.00 "	1 250	"
2.20 P. M.	"	71.94 "	— " 4.32 "	1 850	"
4.00 "	"	70.88 "	— " 1.06 "	800	".

"At 11.20 A. M. water was running clear from 8-in. pipe and reduced in amount over yesterday, with very slight acceleration in flow, when charge was driven.

"Saturday, December 31st, 1898.—3 750 lbs. driven. Plumbed 8-in. pipe as follows:

9.20 A. M.	Distance	68.85 ft.	—Rise 2.03 ft.	2 150 lbs.	clay.
10.20 "	"	68.15 "	— " 0.70 "	1 000	"
11.20 "	"	67.80 "	— " 0.35 "	1 000	"

"Water flowing from 8-in. pipe is clear and reduced in amount over yesterday, shows very slightly the effect of each charge. Total clay driven through 3-in. pipe to date, 26 100 lbs.

"Wednesday, January 4th, 1899.—4 000 lbs. clay driven. Plumbed 8-in. pipe as follows:

12.50 P. M.	Distance	67.70 ft.	1 000 lbs. clay.
2.00 "	"	67.60 "	1 000 "
3.10 "	"	67.55 "	1 000 "
4.45 "	"	67.50 "	1 000 "

"Water flowing from 8-in. pipe has decreased slightly since December 31st; runs clear and shows, very slightly, effect of driving charges. The weight of hammer only, continues to be required to drive clay down.

"Thursday, January 5th, 1899.—5 300 lbs. of clay driven. Weight of hammer only, required; measurements taken on 8-in. pipe as follows:

9.55 A. M.	Distance	67.30 ft.	1 000 lbs. clay.
11.25 "	"	67.22 "	1 000 "
1.05 P. M.	"	67.15 "	1 000 "
2.15 "	"	67.07 "	1 000 "
4.20 "	"	67.00 "	1 000 "

"Saturday, January 7th, 1899.—5 100 lbs. clay driven; no change in measurements taken in 8-in. pipe:

9.10 A. M.	Distance	67.00 ft.	1 000 lbs. clay.
11.25 "	"	67.00 "	1 000 "
1.25 P. M.	"	67.00 "	1 000 "
2.40 "	"	67.00 "	1 000 "
4.10 "	"	67.00 "	1 000 "

"Flow of water from 8-in. pipe clear and constant; shows no effect of charge; driving with weight of hammer only.

"Monday, January 9th, 1899.—6 200 lbs. put in; no change in measurements in 8-in. pipe taken every 1 000 lbs. All show clay at distance from top of pipe of 67 ft., or at elevation—32.44. Flow of water shows decided increase over January 7th, and runs steadily and clear, showing no effect of ramming. Weight of hammer only, required.

"Tuesday, January 10th, 1899.—4 150 lbs. driven. No change in measurement in 8-in. pipe. All show clay at 67 ft. down from top of pipe. Weight of hammer only used, no impact. Water flowing from 8-in. pipe shows slight increase over yesterday; runs clear.

"Friday, January 13th, 1899.—Clay ramming resumed to-day. Total driven, 3 750 lbs. Measurements taken in 8-in. pipe as follows:

10.20 A. M.	Distance,	67.00 ft.	1 000 lbs. clay.
12.00 M.	"	66.90 "	800 " "

"The first 1 000 lbs. drove easily; requiring weight of hammer

only, but driving seemed to stiffen until, at end of next 800 lbs., a slight drop of hammer, about 5 ins., was required.

12.40 P. M. Distance, 66.85 ft.—200 lbs. clay; still slight drop of hammer.

2.30 " " .... —250 lbs. clay. Water in pipe rose suddenly, discharging over edge in quite large volume, indicating clay has been forced upward suddenly; and after this, driving becomes easier, weight of hammer only required at:

2.45 " " 60.90 ft.—Rise 5.95 ft., 250 lbs. clay. Water was discharged as each cylinder full of clay was forced in, and continued to flow between strokes also, at the rate of about 13¼ galls. per minute until 3.15 P. M., when flow stopped, except as charge was driven.

3.50 " " 46.95 ft.—Rise, 13.95 ft. 500 lbs. clay.

4.40 " " 37.90 ft.— " 9.05 " 500 " "

"Saturday, January 14th, 1899.—4 250 lbs. clay driven. Measurements taken as follows:

9.50 A.M. Distance, 35.01 ft.—Rise, 2.80 ft. 500 lbs. clay. Water in pipe rises about 2 ins., when charge is driven, dropping back again to old level, but does not flow out of the pipe. Some water noticed coming up through back-filling around pipe, evidently from leaky joint.

10.45 " " 33.70 ft.—Rise, 1.40 ft. 500 lbs. clay.

11.20 " " 32.90 "— " 0.80 " 500 " "

11.55 " " 32.50 "— " 0.40 " 500 " "

1.10 P. M. " 32.20 "— " 0.30 " 500 " "

2.15 " " 31.75 "— " 0.25 " 500 " "

Driving now began to stiffen up and at 2.45 a slight drop of hammer, 6 ins., was required, continuing for balance of day.

3.10 " " 30.50 ft.—Rise, 1.25 ft. 500 lbs. clay.

3.55 " " 28.95 "— " 1.55 " 500 " "

4.35 " " 28.65 "— " 0.30 " 250 " "

"At this time water in pipe ceased to show any effect of driving charge. This P. M. the clay exhibited remarkable elasticity, sometimes forcing the piston back 3 ft. after driving charge.

"Monday, January 16th, 1899.—Total clay driven, 3 400 lbs. Required impact of hammer to force clay, limiting the height of stroke to 6 ins. About 45 strokes required for the cylinder, which is 6 ft. long. The clay is stiffening gradually and losing some of its elasticity, not springing back as much after each stroke. Measurements in 8-in. pipe as follows:

7.50 A. M.	Distance,	28.55 ft.	250 lbs. clay
8.50 "	"	28.55 "	500 " "
10.00 "	"	28.55 "	500 " "
12.30 P. M.	"	28.55 "	500 " "
1.35 "	"	28.55 "	500 " "
3.50 "	"	28.52 "	500 " "
4.15 "	"	28.65 "	500 " "

"Tuesday, January 17th, 1899.—Continued driving in 3-in. pipe until noon; 1 600 lbs. driven. No rise in 8-in. pipe to-day. Clay gradually stiffening in 3-in. pipe until it requires about 90 strokes, none over 6-in. drop, to force down a cylinder full. Shifted over and started driving in 2-in. pipe at Station 5 + 97.5, 3 R., at 3.35 P. M. Weight of hammer only used, carrying clay down very slowly; 200 lbs. of clay put in.

"Wednesday, January 18th, 1899.—Continued at 2-in. pipe, Station 5 + 97.5, 3 R. 200 lbs. put in, with weight of hammer only. At noon this ceased to have effect, and a few light blows, none greater than 6 ins., were tried. Shifted pile-driver back again to 3-in. pipe in P. M. By means of water-jet, clay was removed from 8-in. pipe to a depth of about 40 ft. in order, if possible, to start clay rising in pipe again when ramming should be resumed.

"Thursday, January 19th, 1899.—Jetted out 8-in. pipe to a depth of 40.8 ft. from top when jet stopped short and seemed to bring up on small spawls or gravel. Resumed ramming at 11.00 A. M. in 3-in. pipe, using about 5-in. drop of hammer. Clay working very stiff, requiring about 240 strokes to drive charge (6-ft. cylinder); 500 lbs. put in. Total rise in pipe, 0.5 ft., from 40.8 to 40.3. Ordered clay driving stopped.

"Friday, January 20th, 1899.—Drove 100 lbs. of clay in 3-in. pipe to-day, somewhat easier than yesterday. This was done for the information of members of the American Society of Civil Engineers who visited the work to-day.

"Saturday, January 21st, 1899.—Resumed jetting in 8-in. pipe in endeavor to increase depth of jetting to at least 20 ft.

"Monday, January 23d, 1899.—Succeeded in jetting out 8-in. pipe to a depth of 60 ft. below top. At this point jet pipe brought up on

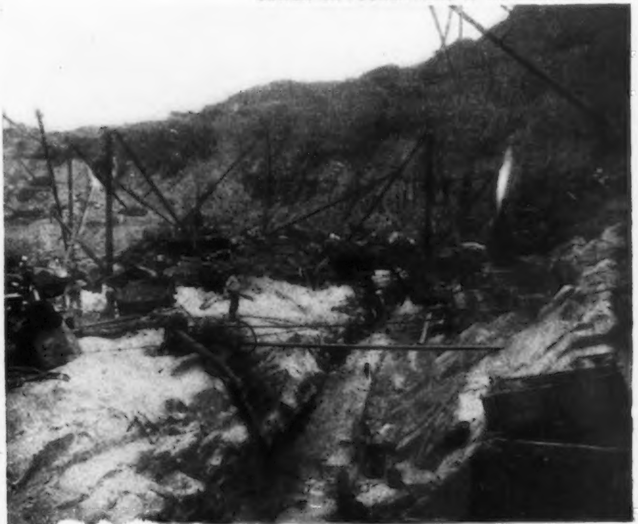


FIG. 1.—AUGUST 10TH, 1897. SEAM AT STATION 4 + 55, LOOKING EAST.

FIG. 2.—OCTOBER 22D, 1897. GENERAL VIEW FROM BERM, AT ELEVATION 115.

what seemed to be a bed of gravel. The flow from the 8-in. pipe increased when this point was reached.

"Tuesday, January 24th, 1899.—Resumed driving in 3-in. pipe; 350 lbs. put in by means of short drops of hammer, none over 6 ins.; 320 short drops required to force down piston (6-ft. cylinder). No rise of clay in 8-in. pipe or any indication of so doing. Ordered clay driveing stopped."

It seemed to be apparent that the cavity at the foot of the 3-in. pipe was well filled with clay, and it was probably due to some obstruction in the 8-in. pipe, such as spawls or gravel which had been forced in with the clay, that there was no longer any rise in the vertical section. The total amount of clay used in this driving is as follows:

2-in. pipe,	Station	6 + 02,	32.4 L.	395 lbs.
2-in. "	"	6 + 03,	17.7 L.	369 "
2-in. "	"	6 + 02,	9.7 L.	1 012 "
3-in. "	"	5 + 95,	12.5 R.	64 775 "
2-in. "	"	5 + 97.5,	3 R.	400 "
Total				66 951 lbs.

66 951 ÷ 113 = 592.5 cu. ft. or 21.9 cu. yds.

The total amount of clay thus driven into the pipes and cavity was nearly 22 cu. yds. The 60 ft. in depth of the outer 8-in. pipe which had been jetted out, was afterward filled with clay and gravel rammed in by hand, and the flow through it was stopped.

The work of tearing out the ruptured masonry due to driving clay through one of the 2-in. pipes was immediately started, and in all about 130 cu. yds. were taken out in following the fissures and cracks until they wholly pinched out. It was assumed in the beginning that the trouble lay in the joint between the upper or surface course of masonry at this point, which had been laid in Portland cement, and the masonry below, which had been laid in slower setting natural cement, and this proved to be the case; as it was found on investigating around the pipe from which the cracks radiated that its upper section and length, of something more than 5 ft. in all, had not been joined or coupled with the section below when placed, and that there was a space of 1 in. or more between the ends of the two pipes, which had allowed the clay, under the influence of the very hard driving, to force its way into the partly set mortar which surrounded the joint.

The course of Portland cement mortar was about 3 ft. thick. It was found that this upper joint of pipe had lifted probably 1¾ ins. from the joint below at the point of coupling. This must have taken place at first when the heavy ramming caused the rupture in that part of the pipe to which the clay cylinder was coupled, and which projected above the top of the masonry. The length of this projecting part was about 1 ft. and the parting of the joints was, of course, at the lower end and about 4 ft. 4 ins. below the masonry surface.

Adjacent to and on the level of this open joint in the pipe was the bed joint of a stone laid in Rosendale cement mortar. The stone was about 5 ft. long, 20 ins. wide and 15 ins. in thickness. The clay was found to have been forced between the under surface of the stone and its mortar bed. The clay bed was about 1½ ins. thick, and, continuing beyond the base of this stone, it rose through the joints along the sides, finding its way then along the top of the course of which the stone just mentioned formed a part, and lifting the course above, which had been laid in Portland cement mortar, and was fairly well set. The first stone mentioned was the only one laid in Rosendale cement that was found to have been disturbed in its bed, and the main crack was everywhere along the junction of the two cement mortars.

The longest horizontal radius through which the clay was found to have worked was about 5½ ft. around the pipe, and it worked up vertically through the mortar joints, and especially along the pipe, about 3 ft. The vertical cracks showing in the masonry surface were traced in some directions for 12 ft., where the width showed about 0.005 ft.; but the upper masonry course was taken out to a considerably greater distance toward and to the up-stream face of the dam, where the seepage of water through the exposed bed joints of one or two stones in this upper course indicated that the horizontal crack had extended with no vertical surface crack above to call attention to it. The clay bed, thus forced under the masonry, was found to be fan-shaped, extending 5½ ft. from the pipe in one direction and about 4 ft. sideways on each side. In the other direction its course was arrested by a large stone which offered no mortar joint that could be followed. The bed varied from 1½ ins. to ½ in. in thickness. It was found to consist of an aggregate of very thin laminations which showed clearly throughout the extent of the bed and indicated the extremely gradual way in which the rupture was produced.

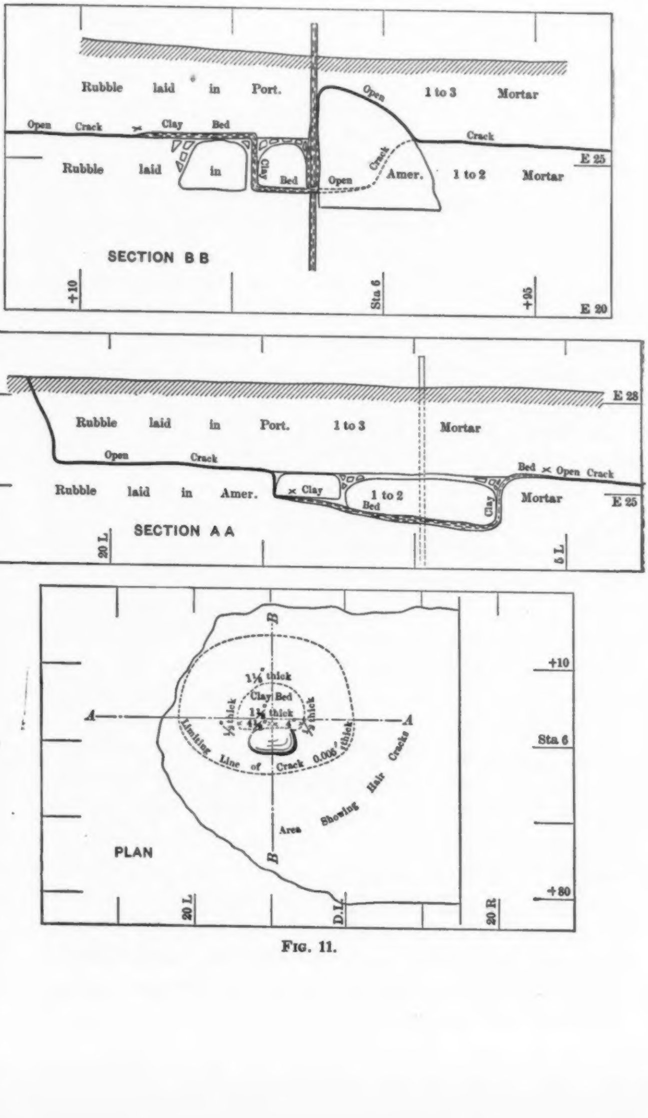

Rubble laid in Port. 1 to 3 Mortar
Open Crack Clay Bed
Open Crack
Crack
Rubble laid in Amer. 1 to 2 Mortar
Clay Bed
E 25
SECTION B B
+10
Sta 6
+25
E 20
E 28
Rubble laid in Port. 1 to 3 Mortar
Open Crack
Bed Open Crack
Rubble laid in Amer. 1 to 2 Mortar
Clay Bed
Clay
E 25
SECTION A A
20 L
5 L
B
+10
A
A
Sta 6
Limiting Line of Crack 0.005' thick
Area Showing Hair Cracks
PLAN
B
+80
20 L
D.L.
20 R

FIG. 11.

Fig. 11 shows in plan and sections the area of the masonry which had to be taken up, as well as the location and extent of the clay bed, the cracks and the particular joints and stones, sketched during the work of rectifying the damage done.

The Main Dam Foundation Masonry.

The laying of the foundation masonry began on May 28th, 1896, in the bottom at Station 8 + 50, as soon as a sufficient area of bottom was ready to warrant it, and by the end of that season nine gangs of masons were at work. This involved the use of eleven or twelve derricks to allow for time lost in shifting derricks as well as for changing gangs from one point to another, owing to frequent changes in the location of sumps and subsidiary pumps which the maintenance of drainage made necessary. In the following season (1897), the number of gangs was increased to 17 on the foundation, as the season progressed, and the total number of derricks in use, including those on the side slopes for passing material from the tracks, was about thirty.

The type of derrick in general use is the "stiff leg" derrick. Having no guys, these derricks did not interfere with the cable service and they were easily moved by means of the cable, without being separated from stiff legs and platforms.

The setting of the first stone in the main dam foundation is shown in Fig. 2, Plate XXXVII. The bottom courses were laid in Portland cement (2 to 1), the vertical thickness of this work, varying from 4 ft. up, depending upon circumstances and particularly upon the amount of seepage through the fissures in the rock and the work necessary to temporarily dam up and divert such flows until the masonry was old enough and high enough to enable them to be blocked off permanently.

The rock bottom was, in all cases, very thoroughly washed and cleaned with brushes and brooms, and was then "painted" with a grout of neat Portland cement, applied with brushes, and which was allowed to set before work was done upon it. This grout was for the purpose of filling all small, fine, open cracks, seams and erosions, which were not of sufficient size or importance to warrant special treatment with the grout pump or box, and about 356 bbls. of Portland cement were used in this way on the main dam foundation and 14 bbls., up to date, on the core-wall and overflow foundations.

Owing to the extreme unevenness of the rock bottom as prepared,

PLATE XLV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 875.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.

FIG. 1.—MARCH 3D, 1899. DAM, FROM HILLSIDE NEAR SPUR ROAD, LOOKING SOUTH.

FIG. 2.—JUNE 22D, 1898. UP-STREAM SIDE, STATION 8 + 70 ±, SHOWING JOINTS RAKED OUT FOR POINTING.

the numerous spring holes which had to be temporarily dammed in order to divert their flows until such time as they could be choked off, and the number of grout pipes to be placed, and in some cases kept in place for months before the grouting could be done, the first season's work on the bottom masonry was done under difficulties, and the general progress up to January 1st, 1897, was between Station 7 + 12 and Station 9 + 62, covering the full width of the dam and varying in depth or height above the bottom from 10 to 40 ft. In all, about 37 000 cu. yds. were laid.

In the following year, the masonry was extended over the whole bottom, with the exception of a narrow strip near the river wall at Station 10 + 00, used as the foundation of a trestle work in connection with the supply tracks, and a comparatively small area in the center of the wall, at about Station 7 + 50, which, during the latter part of the season, was used as a sump-hole for the main pumps, the surrounding masonry forming the sides. During that year the amount of masonry laid in the foundation was about 115 000 cu. yds., and at certain points it had risen to a considerable height, particularly over the points at which the work was started during the preceding season. The width of the foundation to be laid was about 200 ft., and the derricks were arranged in batteries of four abreast across the line of the dam, the plan being to build in racks to a convenient height and then to move the derricks forward in batteries. In this way successive racks or steps were gradually formed for the full width of the work, varying from 17 to 15 ft. in height and from 35 to 40 ft. in width or depth, horizontally, with the derricks moving from the ends toward the middle of the foundation. This arrangement is shown in Fig. 2, Plate XLIV, but, owing to circumstances, it was not until the end of the second season's work that it could be said that the plan had been fully developed and put in complete working order.

The faces or step courses of these racks were limited to rises of 3 ft., with about the same treads, making the slope of the rock nearly 1 : 1. Care was taken to avoid long straight joints across the dam, between successive racks, by varying the lines of their faces at intervals with "scallops" or heavy "returns."

By the end of the second season, the main foundation had risen high enough above the bottom to be drawn in to its neat lines at all points and to afford a parapet which retained the wash from

the earth slopes and enabled the refilling work to be started. On the up-stream side, the masonry face was drawn to its neat lines as soon as practicable upon leaving the rock bottom, though this involved a considerable depth of re-fill between the masonry and rock face, particularly at the deeper points where the rock was disintegrated and had broken back of the excavation lines.

On the down-stream side the masonry toe was built solid to the rock face up to the surface, when it was drawn in to the neat lines planned. This junction with the rock face was made still more compact, as noted in the description of the "grouting," by filling with grout the eroded seams showing in the rock face as the toe masonry was built up.

As above stated, the courses adjacent to the rock bottom were laid in Portland cement, a 2 to 1 mortar mixture being used. The special purpose was to obtain a quick-setting mortar and thus avoid, to as great an extent as possible, any wash or trouble from seepage and flows through the bottom which had been choked off. Above these courses, and for the great bulk of the warm season's work, American cement, mixed 2 to 1, was used. During the winter months, Portland cement, 3 to 1 mixture, was substituted for the American cement, and work was carried on steadily on pleasant days when it was not too cold.

Care was taken to lay no masonry on days when the temperature was steadily below the freezing point, and on cold nights and mornings brine and warm water were used in mixing mortar, and the sand during the whole season was heated and dried in large boxes furnished with steam coils arranged for that purpose. Care was also taken to cover fresh work at night with brine, salt and canvass, and to thoroughly clean its surface and joints in the morning with steam and hot water in order that all frozen dirt and mortar scale might be removed. All stones and spawls used in cold weather were also thoroughly cleaned and washed, and thawed out with hot water and steam; pipes for both being provided for each gang of masons employed.

The stone used for the rubble masonry is quarried from a rocky hillside in the Valley of Hunter's Brook, a tributary of the Croton River, at a point about 2 miles above the dam. This stone is classed by geologists as "gabbro" rock, or, commercially, as a dark colored granite, although it is without quartz and has a large amount of hornblende in its composition. It is very hard and tough, as well as heavy, and

FIG. 1.—MARCH 27TH, 1899. SPILLWAY AND CROTON RIVER. LOOKING EAST.

FIG. 2.—MAY 27TH, 1897. MAIN DAM MASONRY.

weighs 185 lbs. per cu. ft. The quarry is connected with the dam by a railroad, and the stone is quarried and sent down in large blocks varying in size, ordinarily, from 1 to 3 cu. yds., although the greater limit is not reached commonly. Stones even of larger size have been furnished occasionally, but difficulty in handling them renders such sizes undesirable.

The spawls and small "chunks" are furnished from quarries along the line of the railroad nearer the dam, and are of the country rock, a laminated gneiss.

In laying the stone, care is taken to see that each stone has been thoroughly cleaned and washed with water in summer, and with steam in winter. The stone is bedded in a heavy bed of mortar in which flat spawls have been placed to "make up" to such hollows or deficiencies as may be apparent in the bed of the stone. The stone in question is then raised, and the imprint it has made in the mortar bed is used as a guide to complete the necessary making up; additional mortar is then placed over the new bed and the stone is lowered again into place, care being taken to place it exactly as it was before. It is then shaken down by bars placed successively at different ends of the stone until the mortar underneath is pressed out on all sides, when, if it is apparent that it "floats" freely without touching the spawls or stones below, it is allowed to remain. Should there be any doubt about this, however, it is taken up a second time, or as often as is necessary to insure a thorough and tight bedding, well made up and with the minimum of mortar left in necessary to the result wished.

The spaces around these stones are then carefully filled with mortar into which smaller stones and spawls are hammered, care being always taken that no small stone shall be hammered into place unless there is an ample bed of mortar under it. All old work is thoroughly cleaned with brooms and washed with water before fresh work is built upon it. It is also carefully sounded with iron rods to make sure that no small stones or spawls have been loosened in the bed. In cold weather the precautions necessary in building on old work are greater, as mortar more or less frozen and disintegrated is commonly found on the surface and the depth of spawls liable to be loosened is much greater.

The general character and appearance of the masonry in the racks is shown in Fig. 2, Plate XLVI. The stones were of such size that an average rise of 3 ft. in the courses was readily maintained. On the

down-stream side the batter called for by the theoretical sections was obtained by stepping. For this purpose selected stones were of course necessary. The quality and appearance of this work are shown in Fig. 1, Plate XLV. The steps were laid out with rises of from 24 to 30 ins., the latter limit preponderating, and the whole of the step is built outside the neat batter line.

On the up-stream face all joints were raked out 2 ins. in depth and were then pointed up with Portland cement, mixed 1 to 1. This includes also the core-wall and spillway masonry as well as the foundations of the main dam. Fig. 2, Plate XLV, shows a section of the up-stream face of the main dam, in which the joints have been raked out and are ready for pointing.

The foundation masonry laid to date in the spillway was laid in 1895, since which time nothing further has been done. It is shown in Fig. 1, Plate XLVI. The masonry work of the lower part of the core-wall was begun early in the history of the dam work at the extreme south end, and was so prosecuted that by the time the masonry of the main dam foundation had reached the point of junction with the core-wall there were only 100 ft. of wall foundation to be done to complete the connection. This has since been done, and the wall is being built up to the surface of the trench excavated for it.

The refilling against the main dam foundations calls for no special comment except in one instance where, on the up-stream side between Station 6 + 12.5 and Station 6 + 62.5, the bad rock forming the face of the excavation at this point continued to fall into the pit, breaking back of the original excavation lines, after the excavation had reached the bottom, and while the up-stream face of the masonry was building. By the time this masonry had reached a height sufficient to be out of danger from the gradually falling rock, a large mass of the latter had fallen in behind the wall at the bottom into the space which during this time had been used as a sump. It was impracticable to get this material out, as the overhanging rock made it too dangerous for men to attempt it without very thorough protection from above, and it was at the same time extremely desirable to have at this point and elevation especially compact back-filling, rather than a large quantity of loose rock.

The plan and sections in Fig. 10 show the extent to which the rock broke back of the lines and the amount and extent of the broken rock

or debris which fell in from above while the masonry wall was being carried up.

This space was used as a sump-hole, the suction pipe being kept at a low elevation at about Station 6 + 20, at which point the rock slope stood. The water flow through the face of the rock was very free, as the face was full of open fissures, particularly through that part of the face shown in the plan as furthest to the right. This formed quite a recess, and through it came at this time most of the flow pumped from the up-stream side of the dam.

This water flow brought a large amount of fine silt with it, and it was reasonable to suppose that the debris at the bottom and along the wall was, in the course of time, filled with it. It was found impossible to force pipes through the debris to the bottom to test this, and two holes were therefore drilled through the masonry at such positions and angles as to reach low points in the filling. These holes are shown on the plan and sections. Drilling them was a matter of considerable difficulty, but they were finally forced through, but filled immediately with fine silt forced in from below, and no grout could be pumped into them. Two other holes were started in this vicinity, for the same purpose, but could not be carried through.

In making further attempts to prove the compactness or otherwise of this mass of debris the surface shown in the plan to the left of the dotted line was covered with several feet of fine sand and gravel after the pipes indicated had been forced into the mass as far as possible, which in some cases was 5 or 6 ft.

The inflowing water, as stated above, showed itself particularly in the space to the right of the dotted line, in which no gravel was placed. The pump suction back of the wall at about Station 6 + 00 kept the water well below the general surface of the debris which had been covered with gravel, and into three of the eleven pipes 17½ bags of American cement (1 to 1 mixture) were pumped as shown. The other pipes, which were tried later, would take nothing, and the job was finally completed by pouring a large amount of grout, 30 bags of American cement (1 to 1 mixture), into the water space back of the dotted line. This pouring was kept up until the water flow was stopped at the point under treatment, and was forced through other seams in the rock face more directly to the sump-hole which was back-filled later with gravel and sand in the ordinary way.

The Pumping for the Main Dam Foundation.

The work of pumping began in April, 1895, a 10-in. two-cylinder Worthington pump being installed at first. It was placed as near the sump as possible, and was fed from boilers placed at the top of the slope on the down-stream side, not far away. Two 100 H.-P. boilers were installed at this time. This number was increased later to four in all, and at certain times, when the demand for steam for the pumps was heavy, but little outside use was made of the boilers, although the excess in boiler power was at least one, when they were all working at their full capacity. As a rule, however, the whole four were kept continuously in use, working moderately after the 10-in. pump had been replaced, a few months after its installment, by a 12-in. compound, double-cylinder pump of the same make, to which two others of the same size were added later.

With this force of three large pumps two were kept at work at moderate speed, while the third was held in reserve, and the 10-in. pump kept either as additional to the reserve or at times used in connection with a number of smaller auxiliary pumps which were constantly in use during the excavation work and until the foundation masonry was complete, pumping from various points in the bottom to the main sump. From the beginning of pumping operations until November, 1898, the main pumps were kept on or near the lower or down-stream slope of the main cut, and the sump was maintained near by, either on the natural bottom, or, as happened during one winter, in a large hole left in the bottom masonry at a low elevation near the down-stream toe of the dam.

It was extremely inconvenient at times to have to limit main pumping operations to one point and to be obliged to lift all the water from the auxiliary pumps, in some cases over the low-lying portions of recently laid foundation masonry, but the risks to main steam pipes laid across the dam would have been too great, either before or after the beginning of the mason work, while the water flow was large. The discharge was ordinarily through a system of four pipes 12 ins. in diameter, two of which were laid through the lower wing-dam at a low elevation and two through the river wall at a somewhat higher elevation. By this means a considerable lift was avoided, the top of the wing-dam being in the first case about 20 ft. above the pipe openings.

TABLE No. 1.—MEAN MONTHLY TEMPERATURES OBSERVED, IN DEGREES, FAHRENHEIT.

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	APPROXIMATE LOCATIONS OF OBSERVATIONS.					
Date.	$\frac{8 + 50}{195\ L.}$ $\frac{8 + 10}{195\ L.}$	$\frac{6 + 50}{36\ R.}$ $\frac{6 + 00}{30\ R.}$ $\frac{6 + 30}{30\ R.}$	$\frac{7 + 60}{200\ L.}$	$\frac{7 + 70}{30\ L.}$ .... Cave.	$\frac{7 + 00}{12\ R.}$ $\frac{6 + 00}{70\ R.}$	In Channel. River.
	°	°	°	°	°	°
Feb., 1896	44.1	50.2	....	....	....	35.8
Mar., "	42.8	50.8	....	....	....	36.0
Apl., "	45.0	51.0	....	....	....	51.5
May, "	50.0	52.7	....	....	....	....
June, "	51.5	54.0	....	....	....	....
July, "	56.7	53.3	51.3	....	....	82.0
Aug., "	61.0	52.0	60.0	....	....	72.0
Sept., "	63.0	51.5	65.0	62.0	....	73.0
Oct., "	67.0	53.0	66.0	64.0	....	....
Nov., "	64.7	....	60.0	60.0	....	56.0
Dec., "	56.6	52.5	62.8	55.6	57.6	35.6
Jan., 1897	55.0	51.0	57.0	53.0	55.0	34.0
Feb., "	50.0	50.0	50.0	50.0	53.0	....
Mar., "	46.0	48.7	48.7	48.7	52.0	40.3
Apl., "	42.7	48.0	45.3	47.3	51.3	....
May, "	48.0	....	48.0	48.0	51.0	65.0
June, "	64.0	56.0	58.0	54.0	54.0	76.0
July, "	65.0	56.5	64 0	58.5	56.2	76.0
Aug., "	68.7	60.7	64.7	62.0	61.7	76.0
Sept., "	71.2	62.7	71.2	66.2	66.2	74.2
Oct., "	69.6	64.4	69.6	64.8	65.2	64.5
Nov., "	66.5	63.5	67.0	63.0	64.0	47.7
Dec., "	61.6	61.0	63.6	59.2	62.0	40.0
Jan., 1898	56.0	57.0	59.5	57.5	59.5	38.7
Feb., "	46.0	52.0	53.0	52.2	54.7	37.2
Mar., "	43.0	52.2	49.0	51.0	54.4	47.6
Apl., "	45.0	50.7	46.2	49.5	51.7	51.0
May, "	49.2	52.0	49.5	50.0	51.2	60.5
June, "	54.4	54.4	55.4	51.6	50.2	74.4
July, "	61.7	59.0	58.7	53.2	51.0	80.0
Aug., "	64.1	60.0	62.0	56.2	53.5	78.2
Sept., "	68.0	62.2	65.0	59.8	54.4	76.2
Oct., "	68.2	57.0	62.5	60.5	54.0	63.5
Nov., "	68.0	55.3	57.0	....	58.0	49.7
Dec., "	66.0	....	66.0	....	58.0	39.0

Valves at the outer ends of the lower pipes secured the work from the possible danger of back flow when the water was high in the river. In December, 1898, one of the main pumps was placed on the up-stream side of the dam wall, which by this time had been carried all the way across the valley and up to a considerable height above the bottom, and two were kept on the down-stream side, steam for the former being carried across the foundation wall. By this time the amount of water to be pumped had decreased materially, as a large

amount of back-filling had been done on both sides of the dam, and the elevations of the sump-holes had been raised.

During the progress of the main dam excavation it was early noticeable that the flow into the sump-holes seemed to come from particular points on the gravel slopes, following the toes of the slopes down, as the depth of excavation increased. This was noticeable on both the up-stream and down-stream sides, and the flows or springs continued to be identified easily as the work progressed and the outline of the foundation masonry was completed, and the flows or springs on the upper or lower side were separated. These flows were confined to the gravel slopes, through which they came freely, and as the back-filling was gradually raised they were forced back up the slopes to the vicinity of the points where they had originally shown themselves.

At the south end of the main cut and along the sides near the end, where the slopes were nearly all hardpan, practically no water was encountered, although there was a considerable area of gravel and boulder slope under the hardpan near the south end of the side slopes on the "quarters." There was, however, a considerable seepage through the lower half of the very high hardpan slope at the south end which, particularly in winter, through the frost and thaws, caused a good deal of gradual sloughing off of the bank, although at no time was the amount of seepage enough to cause a definite flow from the slope.

A long series of observations of the temperatures, Table No. 1, taken at the points where the flows were best defined, is of interest as indicating, perhaps, some differences in the causes and origins of the various flows observed.

In Table No. 1 are shown six series of observations, including one in the river channel. In Column No. 2 the observations were of a flow on the down-stream side of the main cut, at the point nearest to the up-stream toe of the lower wing-dam. In Column No. 3 the flow observed was near the point on the up-stream side where the heavy flow or spring at Station 5 + 95 was in time developed, and it is assumed that this flow was the same, practically, that showed in the spring when it was reached. In Column No. 4 the observations are of a flow developed on the down-stream slope some distance from the flow in Column No. 2. In Column No. 5 the flow was from the large cave at Station 7 + 70 ± on the up-stream side. The flow of Column

No. 6 was also on the up-stream side, and the observations were taken at first in a large well-hole which was built, temporarily, near the up-stream face of the masonry, and later from the flow which showed outside the masonry line after the well was filled up and the water forced outside of the masonry limits.

The stations at the heads of the various columns show the approximate locations of the points of flow at which the temperatures were taken. In some cases there were variations of location at intervals owing to the shifting of the springs from various causes, such as a deepening of the excavations in the vicinity, or, as in the case of the well mentioned in Column No. 6, some change in the masonry and the channels left temporarily in it. Column No. 7 shows the temperature of the water in the river.

An examination of these observations shows clearly a certain uniformity in the flows from the excavation, particularly in regard to the times of extreme temperatures, which occur in September or October and March or April. It is evident that there was no direct connection with the water flowing in the river, and that the two springs observed on the down-stream side correspond closely, as might have been expected, while the other three springs located on the up-stream side are uniform in showing less extremes in temperature and also a close correspondence with each other. In the river the extreme temperatures shown were in January or February and July.

The flow observed in Column No. 3 shows, however, but little variation during the first twelve months, quite in contrast to the others. The observations during these months were evidently of the water from the heavy spring which, when solid rock bottom was reached, was found to flow from the large erosion at Station 5 + 95, 12 R. The point at which the temperatures were taken in this case was some distance from the spring hole as finally defined, and the water was then piped for some months directly to a subsidiary sump.

Temperatures of the water, however, continued to be taken as it flowed from this pipe and later from the pipe used to divert the spring flow to the up-stream side of the masonry. The elevation of the outlet of this pipe was increased gradually as the masonry and the back-filling rose, and the observations were continued until the clay driving had stopped the flow of the spring. The observations from January, 1897, showed variations which correspond with the variations observed

at the other points of flow, although they were not so marked in degree. As this spring was quite different in its characteristics from any of the others coming originally from a much greater depth and with the location of its flow confined to one place during all the time, it is an interesting question as to why, after a year's nearly steady temperature, variations corresponding with the surrounding springs should develop. It may be that after the first twelve months this spring flow was, to a certain extent, exhausted, and its temperature was affected in a greater degree by the increasing proportion of ground-water near its outlet. Its evident decrease in flow, as time passed, may be one argument in support of this assumption.

A daily record of rainfall, river flow and pumping is shown on Plate XLVII. The rainfall and river gauge readings are shown from September 1st, 1895. The pumping record is begun in October, 1895. This shows irregularities for the succeeding eight months, which are partly due to lack of systematic observations and records during that time, and partly to the frequent changes in the location and elevation of the pumps, as it was during these months that the increase in the depth of the sumps was the most marked. The duration of each rainfall is indicated as nearly as possible by the span of the bracket, the height showing the total fall in inches. The flow in the river is indicated by observations taken at a gauge a short distance below the dam location, and the diagram shows the depths of the flow at that point in comparison with the low-water elevation, which is about at gauge reading 1.70. The pump diagram is based on the average daily speed, in strokes per minute, of one 12-in. pump, and all observations within the above time limits have been commuted to this basis, as the only one by which a direct comparison of the pumping work from time to time could be had.

As to the actual amount of water pumped, various tests of the pump capacity were made from time to time. The 10-in. pump was stated by the makers to have a capacity of 1 500 000 galls. per 24 hours, with a maximum rate per minute of 36 * strokes. Each of the 12-in. pumps, at the same maximum rate of speed, had a capacity of

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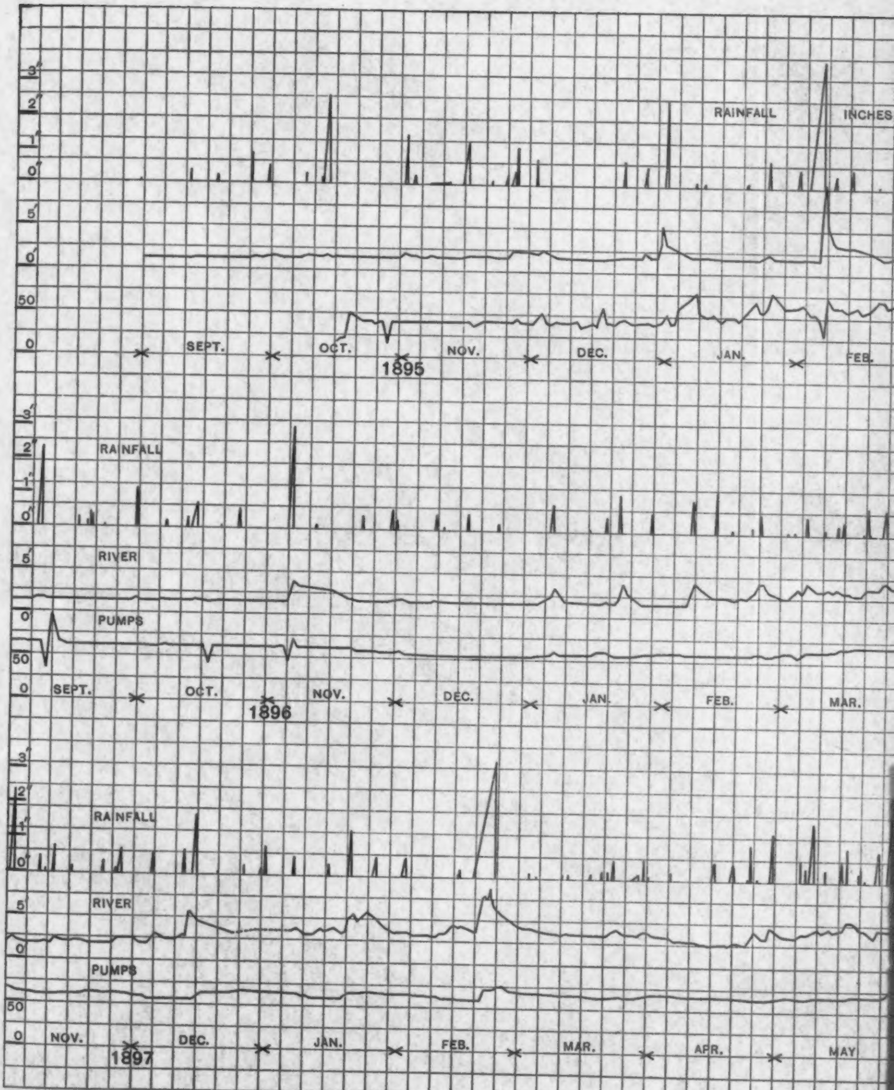
* Double strokes, or one complete revolution, or "cycle," of the pump action. The capacity of each 12-in. pump cylinder for one complete stroke was about 29 galls., the diameter of the piston being 17 ins. and the length of stroke 15 ins. The 12-in. pumps were so designated to agree with the diameter of the discharge outlet.



RAINFALL
INCHES
3″
2″
1″
0″
5′
0′
50
0
SEPT.
OCT.
NOV.
DEC.
JAN.
FEB.
1895
RAINFALL
RIVER
PUMPS
SEPT.
OCT.
NOV.
DEC.
JAN.
FEB.
MAR.
1896
RAINFALL
RIVER
PUMPS
NOV.
DEC.
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MAR.
APR.
MAY
1897

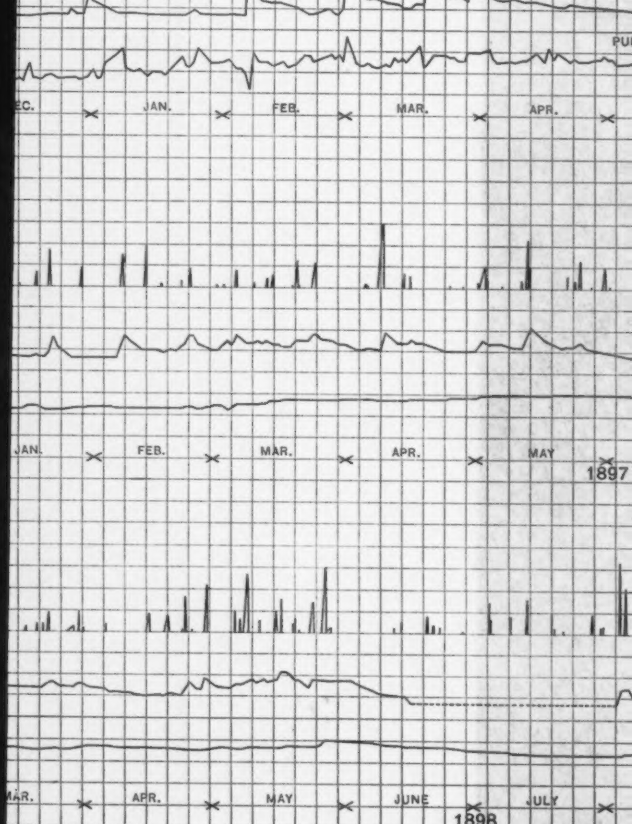
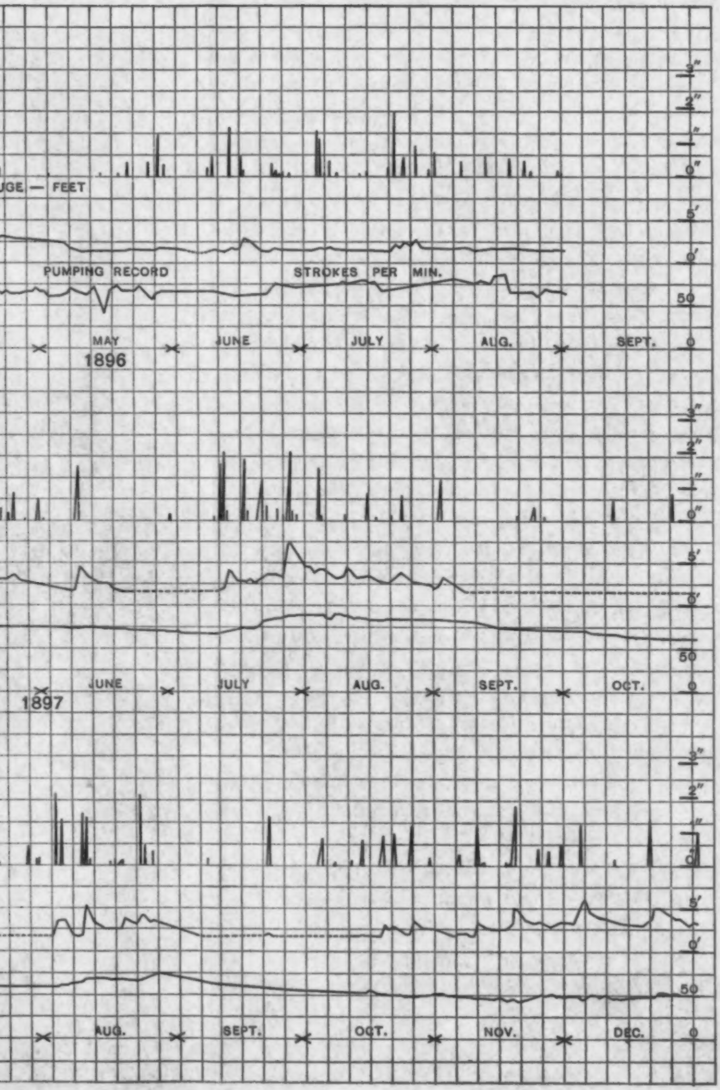

RAINFALL
INCHES
READING OF RIVER GAUGE — F
PU
EC.
JAN.
FEB.
MAR.
APR.
JAN.
FEB.
MAR.
APR.
MAY
1897
MAR.
APR.
MAY
JUNE
JULY
1898

PLATE XLVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 875.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.



4 000 000 galls. per 24 hours. The tests of the actual amount of water pumped are as follows:

On Sunday, October 27th, 1895, water in the main cut was allowed to rise from Elevation 7.81 to Elevation 10.14, the limits between these elevations having been carefully cross-sectioned. The amount of inflow was estimated as 814 275 galls. in 7 hours, or at the rate of 2 800 000 galls. per 24 hours.

The pumps stopped at 8.40 A. M., and resumed work at 3.40 P. M. The water during this interval rose 2 ft. 4 ins., and seemed to rise at a constant rate of nearly 4 ins. per hour.

* The small Worthington pump (1 500 000 galls.) was unable to control the water in the main cut below Elevation 13. The water gained on the pump up to this elevation, but was then held constant by the small pump.

On December 19th, 1898, an experiment on the pumping capacity of one 12-in Worthington pump was made. The result showed a capacity of 50 galls. per stroke.

The experiment was made by the use of a large sump-hole on the down-stream side of the main dam. One side of this sump was formed by the wall of the dam, the others by the back-filling in progress at the time the experiment was made. The following is a *résumé* of the results. The sump had been carefully cross-sectioned.

DECEMBER 19TH, 1898.

Calculations to determine efficiency of pumps and amount of water flowing into sump. 12-in. pump, down-stream sump.

Capacity of sump between Elevation — 2.2 and Elevation — 8.0	287 512	galls.
Experiment 3.20 P. M. to 7.10 P. M. 230 minutes.		
At 3.20 P. M. the sump was empty, pumps shut off.		
By 7.10 P. M. the sump had been filled by water flowing in from springs. In 230 minutes the amount of water flowing in equals capacity of sump	287 512	galls.
Flow per minute	1 250	"

* Note taken at time above experiment was made.

Experiment 10.30 A. M. to 2.27½ P. M.

237½ minutes.

Sump full at beginning, empty at end.

950 815 Gauge reading at 2.27½ P. M.

939 181 " " 10.30 A. M.

11 634 Number of strokes of pump during experiment.

49 Number of strokes of pump per minute.

Flow during experiment

= 1 250 galls. × 237½ = 296 875 galls.

Capacity of sump	287 512	"
Amount pumped	584 387	"
Divide by number of strokes (11 634) and we get	50.23	" per stroke.

Experiment 2.27½ P. M. to 3.20 P. M.

52½ minutes.

Sump empty at beginning and end.

952 453 Gauge reading at 3.20 P. M.

950 815 " " 2.27½ P. M.

1 638 Number of strokes of pump during experiment. Multiply by 50.23 galls. per stroke and we get

82 277 galls. pumped out during experiment equals gallons flowing in

82 277 ÷ 52.5 = 1 567 galls. Flow per minute (when sump is empty)

1 638 ÷ 52.5 = 31.2, average number of strokes per minute.

Flow per minute with sump empty during experiment	1 567	galls.
Flow per minute with sump empty at beginning and full at end, or *vice versa*	1 250	"
Difference	317	"
Capacity of pump per stroke from experiment	50.23	"
Capacity of pump per stroke (pump measurement)	58.00 ±	"

N. B.—For 24 hours previous to these experiments the water in the sump was held at Elevation — 5.54, and the speed of the pump averaged 29.5 strokes per minute. The rise and fall of the water in the sump in the above experiments was between Elevations — 2.2 and — 8.0.

At 49 strokes (double) per minute the amount pumped is at the rate of 3 550 000 galls. per 24 hours. As the maximum number of strokes shown on the pump diagram is not more than 90 strokes per minute, it may be assumed that the maximum flow into the pit was less than 7 000 000 galls. per day. 29.5 strokes per minute equals about 2 160 000 per day.

A special experiment was made at a time when one 12-in. pump was at work on each side of the dam, in order to detect if possible any variation in the pumping rate owing to the relative difference in the elevations of the sump levels. The result is as follows:

TABLE No. 2.—Calculations to Determine Effect of Elevation of Water in Sumps on Up-Stream and Down-Stream Sides of Dam upon Amount of Water Pumped.

Experiment, December 21st, 1898.

Elevation of water in down-stream sump — 6.55.
" " " up-stream sump + 8.7.
12-in. pump used in each sump.

Time.	Down-Stream Sump.			Up-Stream Sump.
	Register.	Difference.	Strokes per minute.	Strokes per minute.
7 A. M.	017 256			
8 "	018 968	1 712	28½	21
9 "	020 726	1 758	29⅓	19½
10 "	022 415	1 689	28	20¼
11 "	024 457	2 042	34	20
12 M.	025 883	1 426	23¾	21
1 P. M.	027 601	1 718	28⅔	21½
2 "	029 306	1 705	28⅓	21¼
3 "	031 030	1 724	28¾	21¼
4 "	032 739	1 709	28½	21¼
5 "	034 462	1 723	28⅔	21½
6 "				21½
Total strokes			286⅓	231
Average strokes per minute during day			28.6	21.0

Water in down-stream sump lowered during night.

Experiment, December 22d, 1898.

Elevation of water in down-stream sump — 8.01.
" " " up-stream sump + 8.7.
12-in. pump used in each sump.

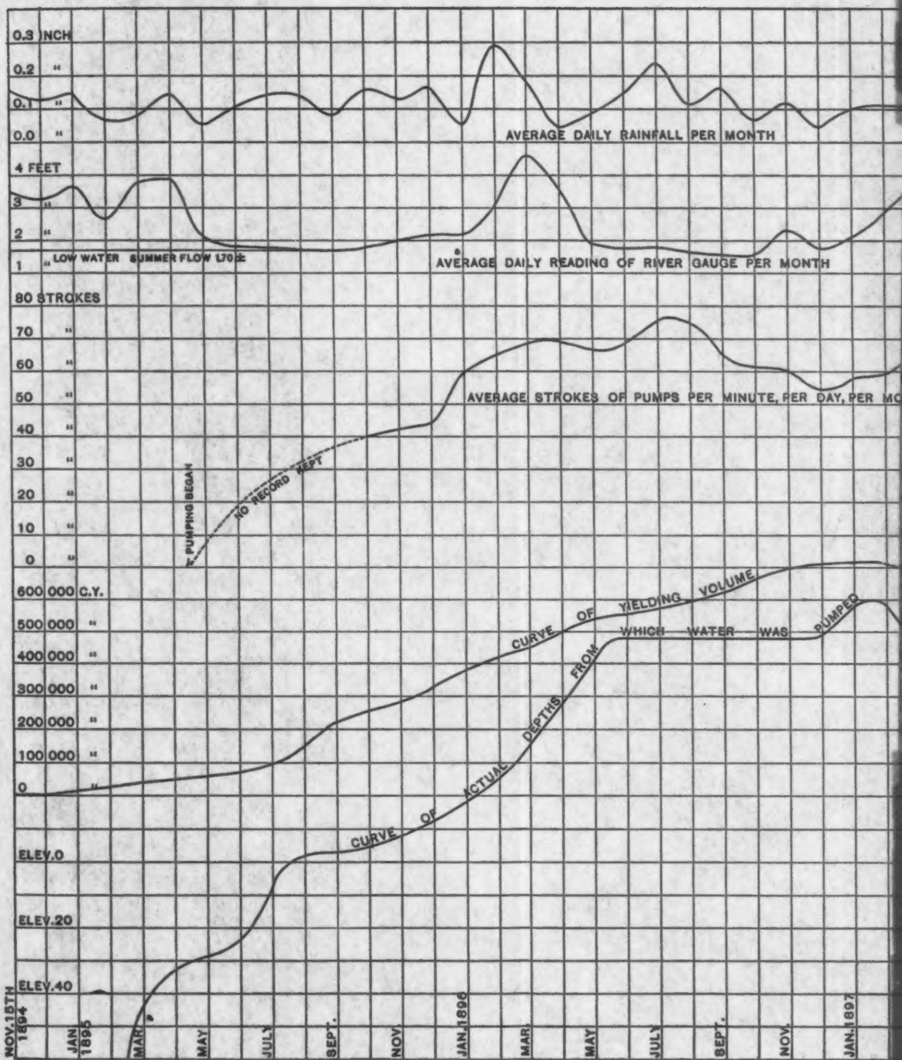
Time.	Down-Stream Sump.			Up-Stream Sump.
	Register.	Difference.	Strokes per minute.	Strokes per minute.
8 A. M.	061 265	....	....	19½
9 "	062 908	1 643	27⅓	19½
10 "	064 589	1 681	28	19½
11 "	066 252	1 663	27⅔	19½
12 M.	067 893	1 641	27⅓	20
1 P. M.	069 535	1 642	27⅓	19
2 "	071 167	1 632	27⅓	19
3 "	072 807	1 640	27⅓	20½
4 "	074 425	1 618	27	19½
5 "	076 046	1 621	27	20
6 "	....	....	....	21
Total strokes			246⅓	217
Average strokes per minute during day			27.4	19.7

Conclusion.—The relative heights of water on the up-stream and down-stream sides of the dam seem to have no effect on the relative amounts of water pumped.

Note.—On December 20th about 0.35 in. of rain fell, which may account for the greater amount of pumping on the 21st than on the 22d.

It is not assumed that the foregoing tests are accurate. They are sufficiently reliable, however, to enable it to be said that the maximum daily pumping did not at any time exceed 7 000 000 galls., and the pump diagram furnishes a reliable comparison between the amount done from time to time and the maximum. These diagrams, it may be said, are chiefly of interest in that they serve to show the relations existing between the rainfall and the resulting river flow and necessary pumping. As to whether any deductions of value can be made, excepting that in a gravel bottom, below sea level, in close proximity to a river large in times of heavy flow, the amount of water pumped was under 7 000 000 galls. per day, while the area of the pump well was at least 3 acres, and the depth 130 ft., remains to be seen.

Plate XLVIII shows curves deduced by averaging per month the various data shown in Plate XLVII. On it are also shown the curve of increasing depth of sumps from which the pumping was done, and a curve showing the increase in yielding volume as the depth of the sumps and the size of the excavation increased. These curves are not extended beyond March, 1898, as by that time the refilling had been



0.3 INCH
0.2 "
0.1 "
0.0 "
AVERAGE DAILY RAINFALL PER MONTH
4 FEET
3 "
2 "
1 " LOW WATER SUMMER FLOW 1.70±
AVERAGE DAILY READING OF RIVER GAUGE PER MONTH
80 STROKES
70 "
60 "
50 "
40 "
30 "
20 "
10 "
0 "
AVERAGE STROKES OF PUMPS PER MINUTE, PER DAY, PER MO
PUMPING BEGAN
NO RECORD KEPT
600 000 C.Y.
500 000 "
400 000 "
300 000 "
200 000 "
100 000 "
0 "
CURVE OF YIELDING VOLUME
CURVE OF ACTUAL DEPTHS FROM WHICH WATER WAS PUMPED
ELEV.0
ELEV.20
ELEV.40
NOV.15TH 1894
JAN. 1895
MAR.
MAY
JULY
SEPT.
NOV.
JAN.1896
MAR.
MAY
JULY
SEPT.
NOV.
JAN.1897

GOW

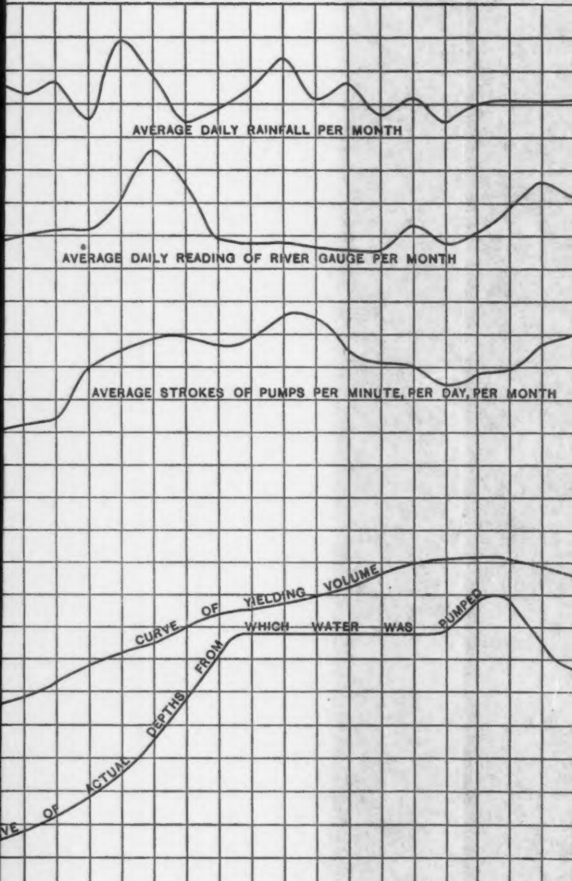
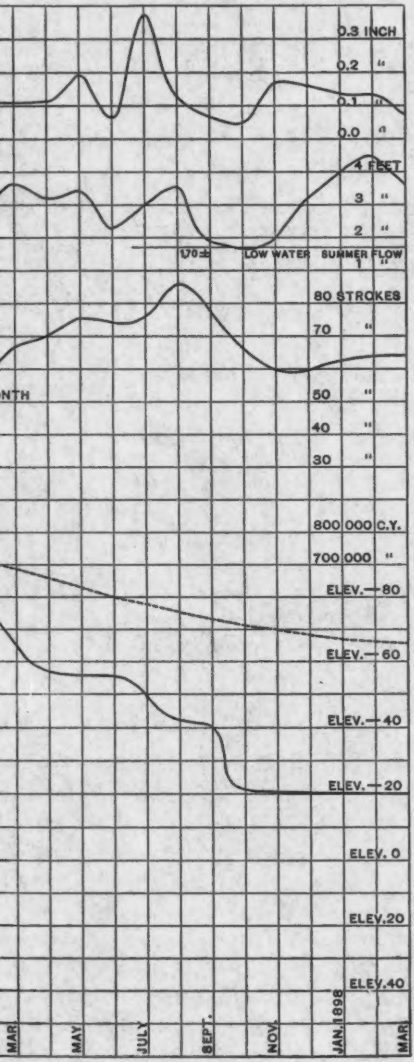

AVERAGE DAILY RAINFALL PER MONTH
AVERAGE DAILY READING OF RIVER GAUGE PER MONTH
AVERAGE STROKES OF PUMPS PER MINUTE, PER DAY, PER MONTH
CURVE OF YIELDING VOLUME
VE OF ACTUAL DEPTHS FROM WHICH WATER WAS PUMPED
NOV.
JAN. 1896
MAR.
MAY
JULY
SEPT.
NOV.
JAN. 1897
MAR.

PLATE XLVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 875.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.



well started, while the whole foundation had been covered with masonry which had been carried up to a considerable height above the bottom.

The curves show, as might have been expected, the effect of the rainfall upon the river which for most of the time is closely corresponding, although the river may be somewhat slower in its action. This does not hold good for the dry summer and autumn months of 1895 and 1896, but the correspondence is close in 1897 when extreme low water was not reached until September. The comparatively heavy flow in February, 1898, however, does not seem to be accounted for by any special rains at about that time.

The pump curve shows a constant rise in 1895 and to May, 1896, as the depth and yielding volume increased. The apparently extra rise in this curve from January to April, 1896, may have been influenced by the rainfall, to which it seems to have responded more quickly than the river. Again, in July and August, with constant depth, there is shown a quick response to the rainfall which is not noticed in the river. In December the pumping had fallen off materially, although the depth and yielding volume were on the increase. The gradually diminishing rainfall, from September to December, may possibly account for this. The steady increase in the rain from January to July, 1897, is noticeable in the river curve and marks a constant increase in the amount of pumping, although the maximum of yielding volume and depth is reached six months earlier. The falling off in pumping from July to December, corresponds fairly, although, perhaps, a month behind in time, with the rain and the river curves, but results from December to March, 1898, are due partly to the influence of the back-filling, which must have begun to make itself felt, and partly, doubtless, to the fact that the yielding volume was beginning, as it did in the previous year, to show signs of being pumped out at the end of the dry season.

It seems fairly conclusive that the flow in the river had, on the whole, but little direct influence on the pumping. In other words, the wing-dams were efficient in stopping anything like a direct leakage or flow from the river to the excavation pit. Another conclusion is that a considerable time elapsed between the rainfall and its effect on the pumps, amounting in certain cases to as much as two months.

The data from which the curve of yielding volume is obtained are due to calculations which show approximately at the end of each two months the amount of sand and gravel on the slopes of the pit which furnishes water storage space. It was assumed that all space in the slopes above an angle of 20° with the horizontal, the vertex being taken at the lowest point in the section excavated and kept clear by the pumps, would yield water, except those parts of the slopes which were formed of hardpan and which were not included in the calculations of volumes.

Fig. 12 shows two diagrams of pumping commuted to equivalent strokes of a 12-in. pump, covering the time from November, 1898, to May, 1899. One of these is the record of the pumping from the up-stream side of the main dam, which was done wholly by a 12-in. pump during that time.

On the down-stream side other pumps were used, as noted on the diagram, but some careful experiments were made by which the relative amount of pump work done has been fairly commuted to the 12-in. standard. The accompanying diagrams, which show in both cases the mean elevations of the two sumps from time to time and at the same time the difference in elevation of the water on the two sides of the dam, certainly do not indicate any connection between the two sump-holes. The up-stream pump shows a very slight increase in March and a very gradual decrease to May 15th, with a gradual rise in water elevation to the middle of February, and no material change later. On the other hand, the down-stream pumping shows a material diminution about February 1st, with many changes of water surface and a very material rise in the sump elevation at about the same time.

The relative elevations of these sumps varied during these months decidedly, the up-stream sump changing from about 17 ft. above the sump on the lower side, to an extreme of about 10 ft. below, most of the change, however, taking place on the down-stream side, as shown. These records are suggestive, in view of the general character of the limestone foundation and the possibility of the presence of open seams and channels below those treated in preparing the bottom for the masonry. With a head of nearly 20 ft., long sustained on the up-stream side, then varied gradually until the head on the other side was 10 ft., there seems to have been nothing, in

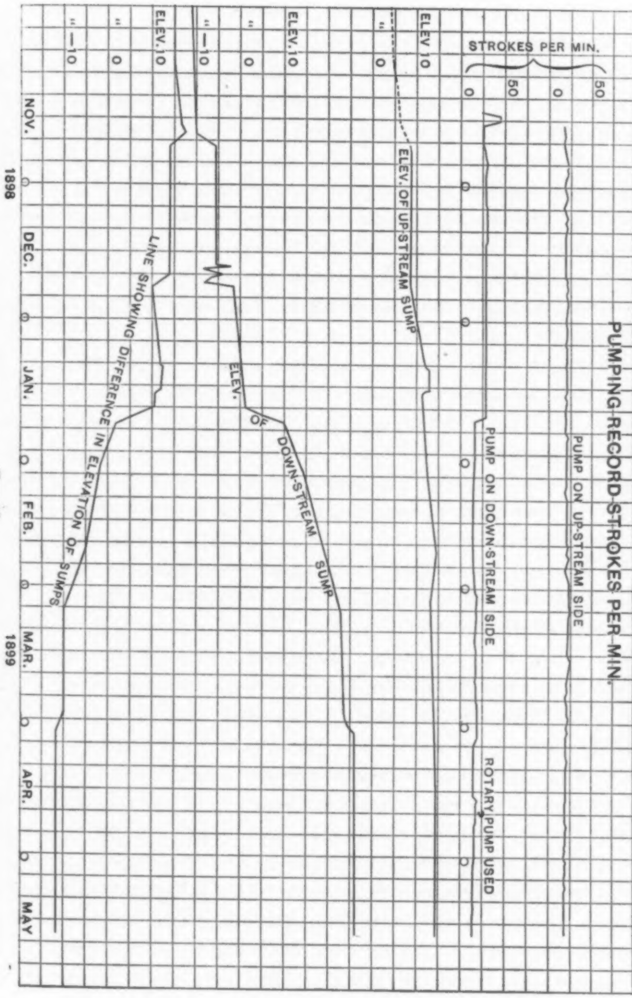

PUMPING RECORD STROKES PER MIN.
STROKES PER MIN.
50
0
50
0
PUMP ON UP-STREAM SIDE
PUMP ON DOWN-STREAM SIDE
ROTARY PUMP USED
ELEV 10
" 0
ELEV. OF UP-STREAM SUMP
ELEV.10
" 0
" —10
ELEV. OF DOWN-STREAM SUMP
ELEV.10
" 0
" —10
LINE SHOWING DIFFERENCE IN ELEVATION OF SUMPS.
NOV.
DEC.
JAN.
FEB.
MAR.
APR.
MAY
1898
1899

Fig. 12.

this reversal of relative conditions, to affect the flow of water on either side.

General Remarks.

While the purpose of this paper is to deal particularly with all the various features of this work which pertain directly or indirectly to the foundations of the dam, dwelling in some cases in considerable detail upon certain points, it is evident that much information, some of it of interest if not of considerable importance, relating to the general design, construction and progress of the work, has been necessarily omitted. In fact, anything approaching a comprehensive account of these matters would require a book of ample size to give the subject adequate treatment. However, a few figures are added here relating to certain prominent features which have not been noted or described previously.

The main feature of the construction work is the rubble masonry. The total amount required will be not far from 650 000 cu. yds., and of this, at the date of writing, about 405 000 cu. yds. have been laid. The contract price for this item is $4.05 per cubic yard when American cement mortar is used. This price is increased to $4.94 and $5.35 per cubic yard, as Portland-cement mortar (3 to 1) or (2 to 1), respectively, is used. The facing stone masonry, of which it is expected that at least 24 000 cu. yds. will be used, forms another important item. This is used for both the up-stream and down-stream faces of the main dam and overflow above the lines or elevations to which the refilling and embankment will be carried. This facing stone is cut in courses which vary in rise from 15 to 30 ins., having a uniform depth of bed and build of not less than 28 ins. Headers, of which every third stone in each course is one, are not less than 4 ft. in length, and are used to insure a bond with the rubble backing or hearting.

All joints of this stone are cut to lay to ½ in. in width from the face back for 4 ins. in depth on the sides and beds. For the remaining depth the stones must be cut full, to joints not exceeding 2 ins. in width between adjoining stones when laid. In this way there is insured a moderately fine outer joint which is thoroughly raked and pointed to a depth of 2 ins. or more, while the wider 2-in. joints give an opportunity for any settlement that may possibly occur in the future due to inequalities between the relative composition of the facing as compared with the backing stone to which it is bonded.

On this facing stone it is depended to insure the practical water-tightness of such parts of the structure as are exposed directly to water pressure. This stone is laid in Portland-cement mortar, (mixed 2 to 1), and in the pointing of the ½-in. face joint Portland cement is also used. As has been stated previously, on all parts of the up-stream face of the masonry, which are planned to be below the back-filling line, and which are formed of rubble masonry, care has been taken to secure well-shaped stones and to fill up the intervening joints very thoroughly with small stones or spawls. This is well shown in Fig. 2, Plate XLV, where the joints are shown raked out and ready for the pointing which, as in case of the facing stone, is done with Portland cement. In this connection, particular attention may be called to the very great amount of refilling which is to be done on the up-stream side, particularly back of the main dam and above the limestone foundation where so much badly fissured rock was found, and which resulted, necessarily, in the great depth of the rock excavation. This refilling, together with the pointing of the up-stream face of the masonry which it covers, is expected to be effectual in stopping percolation through or under the dam, even if in the latter case small open fissures may exist. At any rate, if, as noted previously in the chapter on "Pumping," a head varying from 20 ft. on the up-stream side to 10 ft. on the down-stream side caused no appreciable variation in the pumping of an amount of water, which at that time might have equalled 3 500 000 galls. per day, it does not seem that the head due finally to a full basin can increase very materially such flow as may possibly have already taken place through the limestone foundation rock.

The contract price for the rock excavation is $1.95 per cubic yard. The amount excavated will slightly exceed 300 000 cu. yds. While the price is seemingly a liberal one, it must not be forgotten that, in the bottom work, the blasting and excavation were not done to ordered lines and grades excepting as so directed from day to day, as the only limit in depth was good rock when reached. This necessitated a very great amount of careful hand work, as well as slow and expensive work in finally getting the bottom ready for the masonry.

All the earth excavation work, which has amounted to nearly 1 100 000 cu. yds., was, under the specifications, let at one price, viz., $0.61 per cubic yard, to avoid complication, although, naturally, there would have been little difficulty in separating the amounts lying below

river level, and involving pumping, from the portion remaining. The price was considered fairly low when the risks were taken into consideration. The increase in the length of the main dam, which was determined upon in 1897, resulted in a considerable decrease in the maximum height of the embankment at its point of junction with the main dam. This change has also decreased the amount of embankment as originally planned, and the core-wall trench is now practically wholly in hardpan; the point of junction with the main dam having been advanced to the south until it found the hardpan overlying the bed rock.

The work of construction began in October, 1892, the contract having been let in the previous August. The time limit in the contract was seven years, bringing the date of completion to August, 1899. The extraordinary depths to which the rock excavation had to be made, in order to secure a foundation for the main dam, as well as the change made in the length of the main dam after the work was started, which involved quite an increase in the amount of masonry, justified an extension of time of perhaps one year. The dam, however, will hardly be finished before 1902, making the time of construction ten years instead of eight. This delay is largely due to the dilatory ways and methods adopted in the first two or three years of the construction work, as developments have shown clearly that the plans and methods proposed by the engineers for carrying on the work have so far provided for and anticipated all emergencies and contingencies, in kind if not in degree; nothing unforeseen having happened to materially delay or involve a change of plans beyond the increased depth of rock excavation found necessary and the increase in the length of the masonry dam, as previously mentioned.

DISCUSSION.

E. SHERMAN GOULD, M. Am. Soc. C. E.—Apart from its intrinsic interest, this paper possesses that of timeliness, in that its presentation coincides so nearly with the retirement of Mr. Fteley from the Chief Engineership of the Aqueduct Commission. It describes the progress of what we need not hesitate to style the most important engineering work of the day, from its commencement up to the date of its handing over, fully and successfully launched, by Mr. Fteley to his successor. Mr. Gould.

This very apt connection between the presentation of the paper and Mr. Fteley's retirement will probably strike all members of the Society. To the more limited circle, composed of members and ex-members of the Corps of Engineers of the Old Croton Aqueduct, the death of Julius W. Adams, Past-President Am. Soc. C. E., occurring so near the date of its presentation, lends to this paper an added appropriateness of time and place.

The preliminary studies for the original project, of which Mr. Benjamin S. Church was the author, namely, that of a dam at Quaker Bridge, and of which the present work will be the final outcome, were commenced some twenty years ago under the direction of the late Isaac Newton, M. Am. Soc. C. E., then Chief Engineer of the Croton Aqueduct, by the late E. S. Chesborough, M. Am. Soc. C. E., and the late Colonel Adams. These studies mark the early dawn of the era of scientific high-masonry dam design in this country. The design and construction of what were previously considered high dams, of earth with a center wall of masonry, had already been brought to a high degree of perfection by Tracy, Campbell and Mr. George W. Birdsall, but a masonry dam, upwards of 100 ft. high, was essentially a new proposition. It was new and startling to many of us at that time to learn that the possible crushing of such a structure under its own weight alone, or under its own weight combined with hydrostatic pressure, was a factor of the problem most seriously to be reckoned with. It is probable that this point, and, indeed, the whole question of the profile of equal resistance of such structures, was first brought to the notice of the profession at large through a translation, made by the writer, by direction of Mr. Isaac Newton, of some chapters from Debauve's "Manuel de l'Ingénieur," of which a small edition was printed by the Department of Public Works for the use of its engineer corps, and copies of which found their way to a few other hands. Mention may also be made of two papers contributed about this time by the writer to *Van Nostrand's Engineering Magazine.* Later, the subject was fully developed by Edward Wegmann, M. Am. Soc. C. E., in his masterly and thoroughly exhaustive treatise on "High Masonry Dams."

Mr. Gould. In these studies, the type or concrete idea of the high masonry dam, which with true engineering instinct was seized upon and kept constantly in view, as a safe precedent, by the consulting engineers, was the dam across the Furens, at St. Etienne, France.

But all these particulars are now ancient history, and appeal only to the very limited number of original pioneers in the study of high dams. Their only general interest lies in the evidence which they may afford of the labor and research which characterized the earliest beginnings of the project now being carried out. It is doubtful if any engineering project in this country has ever been made the subject of so much laborious and painstaking study as that described. This paper, taken in connection with the reports of the Chief Engineer, already published, shows us that these studies, taken up by Mr. Fteley where they were left off by the original projectors, were continued by him with unabated zeal and thoroughness.

The description of the system of borings and other explorations given by Mr. Gowen is noteworthy. The juxtaposition of gneiss and limestone, with outcrops on opposite sides of the valley, seems to be characteristic of this and the neighboring territory, and merits study by the geologist. The disappearance and in some cases reappearance of water in the bore holes at great depths is certainly puzzling to account for. Mr. Gowen calls attention to the discrepancy frequently found to exist between the character of the rock as revealed by the actual excavations and that previously predicted from the borings. This discrepancy was also noticeable in driving the tunnels of the New Croton Aqueduct, and it admonishes us that while the diamond drill is of great utility in preliminary explorations, its indications should be taken with considerable reserve, and interpreted very cautiously.

The quotation from Mr. Fteley's Report (pages 483 and 484), in which he recommends abandoning the Old Quaker Bridge location and building a much smaller dam higher up the stream, is an excellent example of sound engineering judgment, and one is rather surprised at the haste with which, in the face of this recommendation, the Commission adopted the Cornell project. The fourth reason advanced by Mr. Fteley for his recommendation seems, however, to require some modification. He says:

"The interest of the money thus saved for the present would, after twenty-five years, represent a large part of the money necessary to then build the higher dam, with the result that the city would then have two dams instead of one for nearly the same expenditure."

This result could only be safely predicated if the amount of interest saved were year by year paid into a sinking fund, and kept intact. This, it is hardly necessary to remark, would be very unlikely to be carried out.

In this connection, a glance at the estimated amount of storage,

consequent upon the completion of the present dam, will be interesting. On page 471 this amount is stated to be 73 236 000 000 galls. The writer has found that a very serviceable formula representing the relation between the total storage required to maintain a desired daily average consumption throughout the year, and the daily average yield of the source of supply, is: Mr. Gould.

$$S = \frac{C^2}{Y} \times 365.$$

In this equation, S = total storage required; C = daily consumption, and Y = daily average yield of the water-shed, all in the same unit. Let us apply this formula to the present case. The capacity of the New Croton Aqueduct is about 300 000 000 galls. per 24 hours, and this may be taken as the maximum daily consumption of New York, furnishable by this supply. The daily average yield of the Croton watershed above the Cornell Dam, will be about 365 000 000 galls. per 24 hours, We would have then:

$$S = \frac{90\ 000}{365} \times 365.$$

$$S = 90\ 000\ 000\ 000 \text{ galls.}$$

This would be about 300 days' supply, as against 244 actually provided, indicating a fair agreement between the two figures.

Although this paper is confined to the foundations of the great dam, it cannot be satisfactorily discussed without some reference to the profile of the dam itself, which is shown in Fig. 2.

The feature of the profile which immediately challenges discussion is its form below the level of the original surface of the ground. It will be perceived that the profile of equal, or approximately equal, resistance is continued down to the bed rock, some 120 ft. below the bed of the stream. The effect of this is to spread greatly the footing course, and to increase correspondingly the immense volume of material to be excavated. Was this necessary, to insure the required stability of the structure ? The writer has no hesitation in saying that both from theoretical and practical considerations he does not believe it to have been necessary or even advisable. He understands that the theory upon which this profile is based is that the dam is, or may be, subjected to the pressure due to a head of water extending from its top, through the excavation and down to the bottom rock on which it stands. He considers this theory as inadmissible. Its acceptance appears to lead to the untenable conclusion that the deeper the foundation, the greater the hydrostatic stress. We have only to consider what a monstrosity would ensue if the dam were carried down to a very great depth, say 300 ft., below the surface and the profile calculated according to this theory. He considers also, that this endeavor to err, if at all, on the side of safety is, to some extent, self destructive, for it

Mr. Gould. results in piling up a huge mass of back-filling upon the outer toe of the dam, adding this extra weight to the already enormous pressure which it is sustaining. The additional spreading of the foot cannot be necessary to resist rotation about the outer toe, because such rotation would be impossible at that depth, nor can it be needed to prevent crushing. That part of the structure which is deeply buried and closely imprisoned on all sides is quite differently and much more favorably circumstanced to resist crushing than is that part which stands entirely above ground with no opposing resistance to prevent the lateral escape of crushed material. In this connection, the writer would quote what he has said elsewhere, as follows:

"Nor can he agree with those who maintain that the thrust of the water from a full reservoir should be considered as that due to a head extending from the top of the dam to the bottom of the foundations. That portion of the dam which is buried in the earth or rock should, in his opinion, be considered entirely apart from the dam proper, and as subject to an entirely different class of stress. He would consider this portion of the structure as forming, in fact, a part of the geology of the territory, and confine his calculations, as regards the thrust of the water, to the superstructure which, standing in relief above the surface of the surrounding ground receives the pressure of the water on one side, and that of the atmosphere only on the other."

Apart from this feature of the profile, some remarks may be made upon the dimensions given to the upper portion. The calculations which lead to the profile adopted were based upon the well-known empirical formulas for resistance to crushing, sometimes called Debauve's formulas, from the fact that they are given by that author in his "Manuel de l'Ingénieur," though not original with him. It is usual to assume that if the profile, so calculated, satisfies the condition of resistance to crushing, it will of necessity satisfy also that of resistance to overturning. This method of calculation results, as regards the latter condition, in a profile which is very light in the upper portion and very heavy in the lower.

In looking at the profile of a very high masonry dam on paper the eye deceives us, unless we keep the scale in mind. We are accustomed to regard such structures—dams and other retaining walls—in reference to their apparent stability as against overturning bodily around the outer edge. Regarded in this way, the profile always seems to be, and indeed is, skimped at the top and redundantly thick at the bottom. This appearance changes, however, when we reflect upon the character of the stress brought to bear upon the base and all the lower portions of the wall. The tendency of this stress is to pulverize the bottom courses, and it can only be resisted by an extension of base beyond what would be required to secure a sufficient moment of resistance to overthrow. To use a homely figure, the dam may be compared to a sack of meal in danger of bursting, rather than to a rigid body in danger of toppling over.

In designing very high dams, therefore, foreseeing as we do the width of base to which we will be forced in order to meet the increasing crushing stress, we instinctively economize in the upper portions, influenced partly, no doubt, by the fact that if failure is to occur anywhere, it had better be at the top than the bottom, and also to make the unit stresses more uniform throughout. But this economy or this desire to equalize crushing stress may be pushed too far. It seems to the writer that this has been done in the present case, where, in his judgment, the outer face of the dam has been hollowed out too much above Elevation 100. Mr. Gould.

It is stated in the paper that the dam has a general factor of safety of 2 against rotation. It is understood that in all calculations the water in the reservoir is supposed to stand level with the top of the dam, or at Elevation 210. This is certainly an extreme assumption. The spillway elevation is 196, its length 1 000 ft., and its required capacity rated at 15 000 cu. ft. per second. This would correspond to an elevation of about 199, or say 200 at most, in the reservoir. It is probable, therefore, that the dam above Elevation 100 has a factor of safety of at least 2.5, and a rough calculation seems to show this to be the case. But it must be borne in mind that these calculations assume a purely static pressure, due to absolutely quiescent water. This dam, however, will act as the retaining wall, or breakwater, of an immense and deep lake, with soundings of 140 ft., in direct contact with an almost vertical back. Over this lake violent storms will rage, accompanied by wave action of tremendous dynamic force. Is this part of the dam sufficiently massive to meet the shock of these waves and hurl them back upon themselves? Hugh fields of floating ice may be expected to thump heavily against the wall; is it heavy enough to resist their impact urged on by wind and waves? To say the least, we must admit that the practical factor of safety is reduced to its absolute minimum.

The writer would be in favor of placing an earthen embankment, well rip-rapped at the back of the dam, for a portion of its height. The effects of such a bank would be to diminish the chance of percolation down the back and reduce still further the bugbear of an exaggerated hydrostatic pressure; to diminish considerably the effect of the deep wave action against the dam, and by maintaining a constant counter pressure against the back, to limit the range of pressure when the reservoir is alternately full and empty. It would be especially advisable to cover the entire area of the refilled excavation within the reservoir with a heavy embankment in order to consolidate by its pressure the material used in refilling. Plate XXXV shows that the inner slope of the earthen dam on the south side already covers, or is to cover, a large portion of this refilling, so that only an extension of the bank is necessary in order to carry out the suggestion.

Mr. Gould. From the start, it was recognized that the chief engineering difficulties to be overcome in the building of the great dam were the diversion of the river and the taking out of the foundation pit. There can be no doubt that the plan pursued to divert the stream was the best, and its complete success, as recorded in the paper, is the just reward of an intelligent design skilfully carried out. There is no doubt, too, that the method adopted for taking out the excavation, by means of an open cut with sloping sides, was the best under the circumstances, and more certain of success than any attempt to shore up the sides could have been. Had the superstructure been designed to rest upon a base with vertical or nearly vertical sides, the suggestion made originally might have been revived; namely, to take out two comparatively narrow trenches, one at the up-stream and the other at the down-stream face of the foundation, sustaining the sides by means of shoring, and building up these two faces first. The central core of earth could then be removed between these two walls, and the remaining masonry laid. Even in this case, however, the surer plan of side slopes might have been found preferable. Be that as it may, the work has been accomplished successfully as described, and at the present time, when the critical period has been safely passed and the foundations brought up to surface level, there can be nothing but congratulations to all concerned in carrying through this bold and brilliant feat of engineering to a triumphant issue.

The successful prosecution of the work below ground depended upon the ability to keep the pits dry. Evidently, this fact was realized fully by the engineers, and a powerful pumping plant installed for the purpose. It is not always thus, and many operations involving deep excavations are increased greatly in cost, difficulty and danger by inadequate and badly managed pumping facilities.

The overflow arrangement seems to have been intelligently planned, and from an examination of the plans and descriptions contained in this paper the writer thinks it would be difficult to find a flaw in the general design of the work, as regards the handling of the water before, during and after construction.

An interesting feature of the work is the earthen dam with core wall on the south or limestone side of the stream. The writer has not noticed any explicit statement in the paper as to why the change was made from masonry to earth, but it may be inferred readily that the limestone rock was not considered sufficiently solid to warrant a masonry dam, pure and simple, for the entire length. If this was the case, then sound engineering judgment was shown in changing over to earth. It may be suggested, however, that the masonry dam might have been continued across the valley, leaving its down-stream face exposed, while the back was protected by the earthen bank. This would have involved more masonry, the cost of which would be

partially off-set by saving the outer wing-wall and outer earthen embankment. Mr. Gould.

In describing the masonry core-wall introduced in the earthen bank, Mr. Gowen speaks somewhat apologetically of its massiveness. In the writer's opinion it errs in the other direction, and should have been considerably thicker than shown. Its top should, by all means, be carried to Elevation 210, the same as the crest of the masonry dam. Carrying the top of the earthen embankment to 220, as shown in the drawings, is excellent judgment.

In the concluding paragraphs of the section describing the protective work (page 487), Mr. Gowen also speaks somewhat apologetically of the great cost of this work. No word of apology is necessary to justify this entirely wise expenditure. Parsimony here would have been the falsest economy.

The account of the manner of carrying on the deep excavations and of dealing with springs and fissures of the rock is very valuable. It appears to have been successful, and in any event is not open to criticism. This work could only be judged on the ground, and while actually going on. At present it is sufficient to know that what was attempted was accomplished successfully.

Something is said on page 497 about a possible upward water pressure against the bottom of the foundations of the dam. The writer considers that all apprehension of danger from such action is groundless. When the small proportionate area which in any event could be exposed to this action is taken into consideration; when capillarity and friction are given their due weight, and when it is remembered that at the worst it would be a hydraulic, not a hydrostatic, pressure that could take effect, as there would always be a line of escape below the dam, it will be realized that this danger dwindles down to a negligible value. The writer would class fears of this nature with those which prompt a continuation of the theoretical profile down to the deep-seated rock.

All the foregoing remarks apply to what may be called, distinctively, the engineering features of the work. The constructional details are also very interesting.

Before commencing to lay the masonry in the foundation pit of the main dam, it is stated that the rock bottom was painted with a grout of neat Portland cement, which was allowed to set before commencing to build. The writer would question the wisdom of interposing a film of cement between the rock and the masonry. He would prefer to bed the stones directly upon the sharp, clean rock.

The arrangement of the derricks and the racking of the work as described and shown on Plate XLIV, Fig. 2, were judiciously planned, and calculated to secure rapid, systematic and substantial work. The cable-way does not seem to figure in this part of the work. It is not

Mr. Gould. stated whether the "Portland" mentioned was American or foreign, nor whether the "American" cement was of the Portland or Rosendale type. This leaves us a little in the dark respecting the relative merits of the American mortar, 2 to 1, and the Portland, 3 to 1.

The stone used under the name of "gabbro" is probably a syenite, differing from granite or gneiss in that the quartz is replaced by hornblende.

The precautions taken in bedding the stones, described on page 525, are those necessary to secure good hydraulic masonry. It is probable, however, that as the work progressed and the gangs became broken in to the requirements of the inspection, the beds were properly prepared at once, without the necessity of raising every stone in order to rebed it. Foremen accustomed only to ordinary first-class masonry, such as bridge abutments, are apt to be dismayed at first by the lavish use of mortar required in hydraulic work. They soon realize however, that, even when mortar is thrown in by the shovelful none need be wasted, for the surplus is forced out by the stones as they are laid, and goes to form the bedding of the neighboring ones. Specimens of the work are shown in Plate XLVI, and to a larger scale in Fig. 2, Plate XLV. In the latter figure some hammer-dressing or possibly "plug and feathering" seems to have been used to secure approximately vertical joints and horizontal beds, and in these respects the work is satisfactory. It must be recognized, however, that the stones are badly shaped for substantial work. Unless they are all headers, which would be bad construction, they are nearly all too high in the rise for the length of bed, making them top heavy and rendering it next to impossible to secure a good bond, as is plainly seen in the figure. In a wall of the immense thickness of the main dam it is true that many defects of this sort are of comparatively little moment, for the wall must be considered in the mass, and besides, true Portland cement mortar proportioned 2 to 1 or even 3 to 1 is a tower of strength, and covers many sins. But in the case of light work, such as the center wall of the earth embankment, the stones should be got out in such shapes that each individual piece is in stable equilibrium when laid in the wall with its best bed down. They should admit of being laid readily, with a perfect interlocking bond, every vertical joint being well capped by the stone above it, the bond being maintained, not only on the face, but throughout the entire body of the work. In Fig. 1, Plate XLVI, showing the unfinished end of the spillway, a tendency may be seen to produce a triple wall, that is, to build the two faces first, and fill in between them afterward. Unless the greatest care be taken to prevent it, thin work will almost always be built this way, but the result is a weak combination, and the tendency should be carefully guarded against.

The prices paid for the different classes of work furnish valuable

economic data. The price of rubble seems very low, although the facilities for using large stones and working generally to good advantage, are very great. The prices paid for excavation, both rock and earth, on the other hand, seem high. No price is stated for refilling or embankment. It is noteworthy that no concrete is mentioned. Mr. Gould.

The over-running of the time limit in a work so carefully planned, and in which there were no blunders to correct, nor extensive additions made after the work was commenced, is suggestive at the present time when other gigantic undertakings are contemplated by the city.

GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—The portions of this paper relating to the borings for determining the nature of the foundations are especially interesting to the writer, as well as the observation that, in the Croton Valley, wherever the bed-rock appeared at one side, it almost invariably dipped down sharply on the other side to a depth at which it would be impracticable to establish a foundation. Mr. Rafter.

This general condition, or a modification of it, is found frequently in streams issuing from the granitic rock horizons of the Adirondack Mountains of New York. The writer has found it repeatedly in his extended series of examinations of sites for storage dams in that region. As to why this phenomenon repeatedly occurs, the geologists are silent; and thus far the writer has been unable to assign any explanation. Certainly, in view of the large amount of high dam construction now projected in the State of New York, an answer to this question would be useful, if for no other purpose than to tell us in many cases what to avoid. An answer is desirable, further, on the general principle that, with the reason fairly understood, we may hope to find more easily the point of minimum resistance—that is to say, the location on a given stream where the conditions are, on the whole, the most favorable.

As to a solution of this problem, the writer considers that undoubtedly it will come through a better understanding of the laws governing glacial drift and the complex phenomena of surface geology, generally.

As stated, some of the Adirondack streams present a modification of the condition described by Mr. Gowen, namely, the bed-rock frequently shows on one side of the valley, dipping down to a few feet below the bed of the stream and then running off on the other side either horizontally or approximately so. This was the condition at Indian Lake, where a dam 47 ft. high was erected in 1898. So far as the studies have been carried, it is the condition at Boreas and Cheney Ponds, Tumblehead Falls, Conklinville and other points proposed as sites for high dams in the Adirondack region, and of which some of the details may be obtained by reference to the writer's reports on the Upper Hudson storage surveys.

Mr. Rafter. In 1893–96, the writer made a series of studies for high dams on the Genesee River ranging from about 100 ft. to 175 ft. in total height. In the work in the Genesee Valley, in 1893, the rocks dealt with were the rather soft and friable Genesee shales. In general terms, the problem was to find material hard enough to carry securely the superimposed weight and, at the same time, insure water-tightness under the foundations and at the ends.

It was deemed desirable to use the diamond drill extensively. The cores taken out showed that the first 20 to 30 ft. of the rock foundation, while evidently capable of carrying the proposed loads, was defective in that there were many minute seams through which considerable water might be expected to escape when under pressure. In order to gain some idea of just how serious a matter this might be, water pressure was applied to drill holes after the drilling was finished, by the use of a rubber packer, placed at different elevations in the holes. Water was forced below the same by means of a pipe passing through the packer. Space will not be taken to describe in detail the arrangements for accomplishing this, because illustrations of such rubber packers may be found in the catalogues of firms dealing in well supplies. Moreover, the appliances and the results attained have been described by the writer somewhat in detail in his reports on the Genesee River storage, for 1893 and 1894. The object in referring to the matter at all on this occasion is chiefly to complete the literature of the subject in the *Transactions* of this Society. So far as the writer is aware, his methods of using water under pressure for testing the quality of rock foundations, as worked out in 1893, were somewhat in advance of methods used previously.*

To illustrate the methods used and the results obtained in 1893, the following abstracts from the log of the tests, as kept from day to day, are presented:

October 17th, 1893.—Tested drill hole *YY4*, using rubber packer set 50 ft. from top of casing. On starting pump, gauge showed 60 to 70 lbs., and pressure rose gradually to 110 lbs. At this pressure the hole took all the water the pump could deliver. After pumping for an hour with the packer at Elevation 539.5, disconnected and found that water ran slowly from top of pipe, about 5 ft. above surface of ground (Elevation 594.0), thereby showing that a small head had been gained at the sides. Packer was then raised to Elevation 559.5 (top of rock at 565.2), and the pump again connected. With the pump at full capacity, the pressure was only 20 lbs. and no more could be gained however rapidly the pump was run. The clear inference is that between Elevations 539.5 and 559.5 there are seams or fissures which allow water to flow out of the drill hole when under about 20 lbs. pressure.

October 19th, 1893.—Tested drill hole *ZZ1*. Packer was first set 19 ft. above bottom (Elevation 532.5). Pump stalled at 100 lbs.

* Report on Genesee River Storage Surveys, Annual Report of the State Engineer and Surveyor of New York, for 1893, p. 416. Also, same report, 1894, p. 360.

pressure. Raised packer gradually, packing it at every few feet by setting pressure above against pressure of water from below. In this way the hole was tested for its entire length and found to stand, as stated, 100 lbs. at the bottom, and from 40 to 50 lbs. in the upper part. Mr. Rafter.

October 31st, 1893.—Tested horizontal hole at foot of Hog-back. Set packer 87 ft. in, or 8 ft. from end. On starting pump, gauge showed 100 lbs. and rose gradually to 140 lbs. Released packer and stopped every 10 ft. until 54.5 ft. from bottom was reached. At this point gauge dropped to 100 lbs., but in 30 minutes advanced again to 140 lbs., and in 15 minutes more to 150 lbs., where it remained for 30 minutes and then dropped to 60 lbs., the steam pressure remaining the same. In 1 hour and 45 minutes the pressure was 40 lbs. At this point water dripped from the side of the Hogback for some distance to the South.

November 2d, 1893.—Repeated the foregoing test with packer 55 ft. from bottom of hole, and with 4 qts. of wheat bran below packer. Pumped with 100 lbs. pressure for 2 hours without effect.

November 2d, 1893.—Tested hole 23 + 42, W. 250, at Hogback location. Hole 84 ft. deep, 14 ft. to rock (elevation of bottom 503.4, top of rock 571.4). Set packer 8.5 ft. from bottom. Gauge showed 165 lbs. Raised packer to Elevation 524.4, and pressure dropped to 80 lbs. Disconnected and put in 4 qts. of bran, whereupon pressure rose to 170 lbs. Raised packer to Elevation 534.0 and still maintained same pressure. At Elevation 536.0 pressure dropped to 100 lbs. and remained at that point even after the addition of 5 qts. of bran. Pressure finally fell to 60 lbs. and remained there for 3½ hours. While pumping this hole, water ran from casing at 23 + 42, W. 350.

November 4th, 1893.—Tested 23 + 42, W. 350, at Hogback location. Set packer 10 ft. from bottom (elevation 518.0), and obtained 40 lbs. pressure. Added 4 qts. of bran without effect on the pressure. In 2½ hours the pressure rose gradually to 65 lbs. Coloring matter was added, and showed in the water flowing from casing at hole 23 + 42, W. 250. On stopping pump, it was found that water pumped into hole had acquired a back pressure of 20 lbs. On disconnecting, water ran from pipe for 1 hour and 49 minutes. This test indicates not only a connection between hole 23 + 42, W. 250, and this one, under the river bed, and independent of it; but also shows backing up, probably in vertical seams, at the sides of the gorge. A number of other tests at this site gave the same result. In one of them the back pressure increased gradually to from 50 to 60 lbs., where it remained stationary during 4 hours' continuous pumping. Water was then discovered running from a fissure in the rock side of the gorge over 100 ft. above the river surface and several hundred feet away.

November 24th, 1893.—Tested B. 40 + 70, W. 750, at Site No. 1. Set packer 6 ft. from bottom. Pressure rose to 180 lbs., when pump stalled. Raised packer 10 ft., or to 16 ft. from bottom, when pressure rose at first to 170 lbs., but in a few minutes fell to 120 lbs., where it remained for 10 minutes, and finally fell to 100 lbs. Uncoupled and added bran, when pressure rose from 100 to 120 lbs. Raised packer to 26 ft. from bottom and gauge showed 40 lbs. Again added bran and gauge rose to 50 lbs. With packer at this elevation gas issued from casing at hole B. 40 + 70, W. 850. Upon raising packer 2 ft. more, larger quantities of gas flowed from the hole. The packer was raised and lowered several times with like results, showing a connection between the two holes at about Elevation 548.0.

Mr. Rafter. The use of wheat bran, as referred to in the foregoing, was for the purpose of determining whether or not the seams permitting the escape of water were of minute or open texture. When minute, the fact was shown quickly and easily by the use of a very small quantity of bran.

In 1896 the final studies for the Genesee storage dam at Portage were made. Here very simple conditions prevailed. The rocks dealt with were the comparatively hard sandstones of the Portage group, and a few borings followed by water-pressure tests removed all uncertainty as to the nature of the foundation. The writer has no doubt that diamond-drill cores and a series of water pressure tests properly carried out can be made to yield more for a given expenditure than any other method of investigating this specific problem thus far devised. Indeed, there is really no other satisfactory method of investigation.

In view of the interesting and valuable results obtained, even from observation of loss of drill water at the New Croton Dam, it seems unnecessary to dilate at length on the practical value of such tests.

The method of stopping cavities by forcing in plastic clay is interesting and undoubtedly new to most engineers. Mr. Gowen is fortunate in the considerable number of either new, or substantially new, details developed on the work under his charge, and which he has presented in this paper.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This paper is a valuable contribution to practical knowledge on masonry dam building, particularly where the bed-rock at the site is of inferior character. It contains many interesting data relating to the treacherous nature of limestone formations. Every student of the stability of masonry dams, however, will feel a certain amount of disappointment in the fact that systematic efforts were not made to determine the amount and extent of the expected up-lift on the base of the dam, due to the upward pressure of the ground-water when the lake is filled.

It will be remembered that during the building of the Vyrnwy Dam Mr. Deacon went to much expense and made many useful experiments with the view of determining the amount and extent of up-lift on the base of the dam, and the results were both instructive and thoroughly convincing.

The mammoth dam now being built on the Cornell site, based on a seamy bed-rock full of running water, certainly furnished rare opportunities for making further valuable experiments in the same direction, and it seems strange that some efforts were not made to get more extended information on this all-important subject, and at no great additional expense. It is extremely doubtful how much of this up-lift can be suppressed by grouting wet seams and filling up cavities in the limestone bed-rock. Where hydrostatic pressures are great, as they will

be in this case, it is hard to understand how the up-lift will be confined to the mouths of the bed-rock fissures exclusively. Mr. Le Conte.

The author states that a great many test holes were drilled and piped, the ground-water in some rising 83 ft., with the lake as yet empty.

It is to be hoped that some of these pipes have been, and will be, maintained and continued up vertically through the completed dam, with the view of noting the changes in the pipe water-levels as the lake fills up.

J. L. Power O'Hanly, M. Am. Soc. C. E. (by letter).—The Croton Dam is a lasting monument to the engineering skill of the expiring years of the nineteenth century. It will remind future generations that American engineers of this age and nation could not only conceive, but execute, great and daring projects. Mr. O'Hanly

In venturing on a brief and desultory criticism of the foundations of this stupendous structure, it appears to the writer that the engineer, in his extreme caution in preparing the rock foundations, leant rather too much toward the side of timidity.

The writer can see no good reason to justify any excavation below Elevation — 25. The testimony of Professor Kemp seems conclusive that little danger need be apprehended from subterranean caves, fissures or springs.

There is nothing in the author's diagnosis to indicate any chemical or mineral difference in the ingredients or constituents of the "hard white rock" and the "soft white rock," except the relative quality of hardness. "Soft white rock" may be, and probably is, "hard white rock" in one of its stages of growth or decay. It may be inferred that either all the "soft white rock" will grow, mature and develop into "hard white rock," or that the latter will deteriorate to the state of "soft white rock." In either case the precaution would be futile.

Have any tests been made to determine whether this "soft white rock" has, to any appreciable extent or to any extent at all, been compressed by the maximum pressure caused by the dam? If the rock stood this test the writer would consider it perfectly safe, by analogy, viewed simply and solely as one of the strata, natural or artificial, constituting the geological formation of that locality.

The following are a few of the reasons which have led the writer to these conclusions. Happily for critics, they are absolved of much of the responsibility which appertains to the executive.

Rankine's notation has, from its familiarity, been followed in these formulas and calculations. The masonry is taken at 172 lbs. per cubic foot, and was calculated as follows: The stone weighs 185 lbs. per cubic foot, and the mortar, which comprises about 17% of the mass, weighs 110 lbs. per cubic foot. The "dirt," overlying the up- and down-stream faces of the dam, is taken at 120 lbs. per cubic foot. The dimensions have been obtained chiefly from Fig. 2.

Mr. O'Hanly. With origin of co-ordinates at O, the extreme point of the masonry of the up-stream face at bed-rock, Elevation — 25, the axes of co-ordinates are Ox and Oy, the former horizontal, the latter vertical. The extreme width of the dam at bed-rock, Elevation — 25, is taken at 206 ft.

S_1, S_2, etc., are masonry sections; S'_1 and S'_2 are sections of the superincumbent earth on the down- and up- stream faces, respectively, of the dam. W is the weight in pounds of a section of the dam 1 ft. long. Wx is the moment in foot-pounds around Ox, and Wy around Oy, and Σ the sum.

TABLE No. 3.—WEIGHTS AND MOMENTS OF DAM.

No. of Section.	Between Elevations.	Σ W.	Σ Wx.	Σ Wy.
S_1	−25 and 00	819 000	9 959 000	79 771 000
S_2	00 " 20	562 000	19 558 000	49 906 000
S_3	20 " 40	489 000	26 748 000	39 902 000
S_4	40 " 60	411 000	30 702 000	30 167 000
S_5	60 " 80	334 000	31 630 000	21 743 000
S_6	80 " 100	267 000	30 598 000	15 726 000
S_7	100 " 120	205 000	27 573 000	10 947 000
S_8	120 " 140	151 000	23 330 000	7 203 000
S_9	140 " 160	112 000	19 544 000	4 738 000
S_{10}	160 " 180	84 000	16 338 000	3 217 000
S_{11}	180 " 200	67 000	14 372 000	2 345 000
S_{12}	200 " 210	31 000	7 230 000	1 085 000
S'_1	−25 " 68	508 000	31 496 000	89 256 000
S'_2	−25 " 68	89 000	5 518 000	475 000
		4 129 000	294 616 000	356 481 000

$x_o = 71.35$. $y_o = 86.3$.

These are the co-ordinates of the center of gravity. The center of pressure falls well within the middle third of the foundation, which insures stability of position.

The area of a unit section of the foundation is 206 sq. ft. = 29 664 sq. ins.

The pressure on the foundation, caused by the weight of the dam and the superincumbent earth, is 140 lbs. per square inch.

The crushing stress,* in pounds per square inch, ranges from a maximum of 16 893 lbs. for Grauwacke, 8 528 lbs. for compact limestone (strong) to 3 050 lbs. for magnesian limestone (weak). The pressure exerted by the dam on its foundation is barely one-twenty-second of the crushing stress of this weakest substance bearing the name of rock.

The Pressure of the Water Against the Dam.—The weight of 1 cu. ft. of water $= w' = 62.4$ lbs., and it is assumed that the crests of waves may occasionally reach the top of the dam. Hence the maxi-

* Rankine's "Manual of Civil Engineering," 7th edition, page 361.

Mr. O'Hanly.

mum height of the column of water, x, is 142 ft. The inclination from the vertical of the imaginary line joining the top of the dam and the bottom of the column of water is $j = 4$ degrees.

The pressure of a unit section of the column of water against the dam is

$$P = \frac{w^1 x^2}{2} \sec j = 31.2 \times 20\,164 \times 1.0024419 = 631\,000 \text{ foot-pounds.}$$

Table No. 4 shows the weights and moments of that part of the dam above the bed of the reservoir, and resisting the direct thrust of the water. The masonry, as before, is taken at 172 lbs. per cubic foot. The notation is the same. S'_1 is the column of water resting on and supported by the curved outline of the face of the dam.

TABLE No. 4.

No. of Section.	Between Elevations.	ΣW.	ΣWx.	ΣWy.
S_1	68 and 80	192 000	1 113 000	12 192 000
S_2	80 " 110	267 000	5 767 000	15 726 000
S_3	100 " 120	205 000	8 508 000	10 947 000
S_4	120 " 140	151 000	9 287 000	7 203 000
S_5	140 " 160	112 000	9 128 000	4 738 000
S_6	160 " 180	84 000	8 526 000	3 217 000
S_7	180 " 200	67 000	8 161 000	2 345 000
S_8	200 " 210	31 000	4 247 000	1 085 000
S'_1	68 " 210	44 300	4 194 000	930 000
		1 152 300	58 951 000	58 383 000

$x_o = 51.16$. $y_o = 50.67$.

These are the co-ordinates of the center of gravity of that portion pressed against by the water.

The condition of stability of friction at any bed joint—the joint at the bottom of the reservoir at Elevation 68—is that the ratio of the horizontal component of the water pressure to the sum of the weight of the dam above that joint and the vertical component of the pressure of the water shall not exceed the tangent of the angle of repose of the masonry, usually taken at about 0.74.

This ratio, in symbols, is

$$\frac{P \cos j}{W + P \sin j}$$

which reduces to

$$\frac{w^1 x^2}{2\,W + w^1 x^2 \tan j} = \frac{1\,286\,300}{2\,304\,600 + 90\,000} = 0.54 \leq 0.74.$$

Mr. O'Hanly. Overturning moment:

$$M = \frac{w^1 x^2}{2} \sec j \left\{ \frac{x \sec j}{3} - (q + \tfrac{1}{2})\, t \tan j \right\}$$

$$= \frac{w^1 x^3}{6} \sec^2 j - w^1 x^2 t \left(\frac{q}{2} + \tfrac{1}{4} \right) \tan j$$

$$= 27\,596\,000 \text{ foot-pounds.}$$

The arm of the equivalent couple is $\frac{27\,596\,000}{1\,152\,300} = 24$ ft., which transfers the point of application of the force 24 ft. to the left of the vertical from the center of gravity.

This leaves the center of resistance, F, 59 ft. back from the face, at Elevation 68, or 40 ft. from the toe of the dam, and well within the middle third of the joint.

The Joint at Elevation 140.—The center of gravity of that part of the dam above Elevation 140 is found as shown in Table No. 5:

TABLE No. 5.—WEIGHTS AND MOMENTS.

No. of Section.	Between Elevations.	ΣW.	ΣWx.	ΣWy.
S_1..................	140 and 160	112 000	1 064 000	4 738 000
S_2..................	160 " 180	84 000	2 478 000	3 217 000
S_3..................	180 " 200	67 000	3 337 000	2 345 000
S_4..................	200 " 210	31 000	2 015 000	1 085 000
		294 000	8 894 000	11 385 000

$x_o = 30.25$. $y_o = 39$.

These are the co-ordinates of that part of the dam above Elevation 140, the vertical from the center of gravity cutting the joint nearly in the center of the figure.

To Find the Center of Resistance of the Pressure against that Part of the Dam above Elevation 140.—The condition of the stability of friction at the bed joint at Elevation 140 is that the ratio of the horizontal component of the pressure of the water to the sum of the weight of the dam above that joint and the vertical component of the pressure of the water shall not exceed the tangent of the angle of repose of the masonry, as explained already. The ratio is

$$\frac{P \cos j}{W + P \sin j},$$

which reduces to

$$\frac{w^1 x^2}{2\,W + w^1 x^2 \tan j}.$$

But the up-stream face of this part of the dam is vertical. Therefore $\tan j = 0$, and the above expression becomes Mr. O'Hanly.

$$\frac{w^1 x^2}{2\ W} = \frac{62.4 \times 4\,900}{2 \times 294\,000} = \frac{306\,000}{588\,000} = 0.52 \leq 0.74.$$

This insures stability of friction at that joint.

Overturning moment:

$$M = \frac{w^1 x^2}{2} \sec j \left\{ \frac{x \sec j}{3} - (q + \frac{1}{2})\ t \tan j \right\}$$

With face vertical, $\sec j = 1$, and $\tan j = 0$, which reduces to

$$M = \frac{w^1 x^2}{2} \times \frac{x}{3} = \frac{w^1 x^3}{6} = 10.4 \times 343\,000 = 3\,567\,000 \text{ foot-pounds.}$$

The arm of the equivalent couple is

$$\frac{3\,567\,000}{294\,000} = 12 \text{ ft.}$$

The vertical from the center of gravity intersects the joint 13 ft. from the face. The center of resistance, *F*, is shifted to the left, or 25 ft. from the face of the dam, being 12 ft. from the toe, slightly outside the middle third of the joint. This seems to be a weak spot in the dam.

On page 248 of Rankine's "Manual of Applied Mechanics,"* occurs the following:

"*Example III.—Triangular Wall with Vertical Axis.*—When the wall stands on a soft foundation, it may be desirable in some cases so to form it, that the center of resistance, *F*, shall be at the middle of each joint, and shall also be vertically beneath the center of gravity of the part of the wall above the joint. In this case, the point of intersection, *A*, of the lines of action of the pressure and weight must also fall in the middle of each joint. To fulfil these conditions, the vertical section of the wall should be an isosceles triangle, the outer and inner faces forming equal angles j on opposite sides of the vertical axis of the wall, and the angle j should be such that a straight line perpendicular to $O\ D$ at C shall bisect the base; that is to say,

$$\text{"}\frac{t \sin j}{2} = \frac{x \sec j}{3}$$

"but

$$\text{"}\frac{t}{2\,x} = \tan j$$

"whence we have

$$\text{"}\sin^2 j = \frac{1}{3};\ \cos^2 j = \frac{2}{3}$$

$$\text{"}\tan \frac{t}{2\,x} = \sqrt{\frac{1}{2}} = 0.707;\ \text{and } j = 35\tfrac{1}{4}^{\circ}$$

"so that the base of the wall is to its height as the diagonal to the side of a square."

How is this to be interpreted? Is it to be interpreted literally? That a dam of whatsoever height, on any foundation, is stable with

*Fifth Edition. London, 1870.

Mr. O'Hanly. dimensions of these proportions? Or is it only applicable within the narrow limits of a low dam? Would the Croton dam, executed on these lines, be safe and stable?

It is assumed that to prevent percolation a wall of adequate thickness at the upper face would needs be built down to bed-rock, that the original earth surface be excavated 8 ft. deep, to be overlaid with a bed of concrete 6 ft. in depth, with the dam masonry and wall connected with an impervious layer of concrete.

The base of the dam masonry would be at about Elevation 40, with a height of dam of 170 ft. Therefore, by the rule, the width of the dam at the foundation would be $170 \times 1.414 = 240$ ft.

But if the dam were founded on the "restored surface," the width at the foundation would be 204 ft. Would this be practicable?

The thanks of members are justly due to the author for this valuable and interesting paper. It would add much to its value if the author could conveniently supplement it with additional information, such as the average monthly strength of the skilled and unskilled force, the quantities of materials and cost of the several works mentioned, the current rate of wages, and a detailed statement of the kind of plant in use and its cost.

Mr. Gowen. CHARLES S. GOWEN, M. Am. Soc. C. E. (by letter).—In presenting his paper on "The Foundations of the New Croton Dam," the writer did not anticipate that the question of the section adopted would be raised as the principal point of one of the discussions offered, and he would, for obvious reasons, prefer to leave the further discussion of this point to those who were originally more interested than he in the matter. Nevertheless, the following is offered in reference to this question, in connection with his reply to the other points raised by Mr. Gould.

As the writer understands it, the following were the principal conditions governing the design of the section:

The water level when the basin was full was taken at Elevation 206.

Water pressure was assumed to obtain to the level of the bed-rock surface.

The back pressure due to the water on the down-stream side (water-table level) was also taken into account and allowed for.

Pressures (calculated) were limited to 15 tons per square foot at the base of the structure (rock surface), and the lines of pressure were kept well within the middle third of the section at any assumed level.

The above conditions are stated because Mr. Gould seems to have been in error in understanding that, in the calculations, Elevation 210 was assumed as high-water mark and that the profile of equal resistance was continued down to bed-rock, 120 ft. (instead of 75 ft.) below the bed of the stream; while, as to the elevation of the overflow (196), provision has been made for an increased height by means of flash-

boards at some future time, and a high-water elevation of 206 is not improbable. Mr. Gowen.

No additional thickness of section was made on account of possible ice pressure or wave motion. It is fair to assume that the extensive overflow located in close proximity to the main dam will operate at ordinary high water to relieve the lake of extensive areas of ice nearly as effectively as if it were on the main dam proper, while the width of the structure at the top (Elevation 210), necessary for a roadway, gives additional weight and stability to the section immediately below.

Mr. Gould maintains, apparently, that no account need be taken of pressures below the restored natural surface, below which the foundations may lie, and that no spreading of the "foot" should take place below this level. In other words, that the structure should be designed only with reference to its height above the restored natural surface; that the base on which it rests should be limited by vertical sides; and that dependence should be placed upon the weight of the refill for resistance to crushing and overturning, both of which tendencies would necessarily be increased greatly by the narrow foundation width. In support of this assumption he has stated that "he would consider this portion of the structure as forming, in fact, a part of the geology of the territory," which would seem to mean that the foundation wall must be taken as equal to the ledge rock below, at least in its capacity to resist crushing strains, and that the refilling or restored material must be as compact as it was originally, before excavation was made, in order to be as effective as possible against tendency to overturn. Is not this assumption extreme?

Further on, Mr. Gould states that the tendency of the stress, on the base and lower portions of the wall of a high masonry dam, is to pulverize the bottom courses, and that it can only be resisted by an extension of the base beyond what would be required to secure a sufficient moment of resistance to overthrow. He then compares such a section, perhaps not inaptly, to a bag of meal in danger of bursting. When, however, he proposes to extend such a section for an indefinite distance below the ground surface, and depends upon the refilling to counteract the bursting and overturning tendencies, it would seem to the writer that the plan is somewhat analogous to that of packing the bag of meal in shavings to avoid further chance of rupture to the bag.

It may be of interest to compare results obtained by Mr. Gould's formula for determining the required storage, with results derived by other methods, and if in case of the New York City water supply we assume $C = 280\,000\,000$ galls. per day—which is all that the Croton is calculated to supply in dry years—and $Y = 360\,000\,000$ galls. per day, which may be taken as a fairly conservative estimate of the average

Mr. Gowen. yield of the water-shed, as deduced from observations extending from 1870 to 1894, inclusive, we have as the required storage—

$$S = \frac{280\ 000\ 000^2}{360\ 000\ 000} \times 365 = 79\ 490\ 000\ 000 \pm \text{galls.}$$

Deductions made on the basis of the Sudbury River records from 1875 to 1895,* inclusive, would give the following as the required storage for a dry year supply of 280 000 000 galls. per day—

Area of Croton water-shed 361 sq. miles.

$$\frac{280\ 000\ 000}{361} = 776\ 000 \pm \text{galls. per square mile, average yield}$$

per day required.

This requires a storage per square mile of water-shed of 200 000 000 galls. to prevent deficiency in a dry year, or a total storage of $200\ 000\ 000 \times 361 = 72\ 200\ 000\ 000$ galls.

Later computations of the yield of the Croton, in which the gaugings of flow have been continued nearly up to date, show a continued close approximation between the actual average daily yield and the required storage as deduced through the medium of Mr. Stearns' tables based on the Sudbury flow, and the actual storage planned.

The question of trenching the outline of the foundation in order to save the excavation of self-supporting slopes was fully considered, in connection with the general problem presented in the matter of the earth excavation at the dam, and the writer is of the opinion, judging from the experience had with this part of the work, that neither time nor expense could have been saved by such methods. The maintenance and drainage of trenches sufficient for the purpose, which on the limestone side of the foundation would have had to be carried to varying and great depths into the bed-rock, would have proved especially expensive, as well as tedious, and, even with the trenches established and the retaining walls built in them, the resumption of the interior excavation work would have tended to cause delay and confusion, as the excavation and masonry work would have had to be carried on in close proximity in a place where lack of working room was always found to be a hard condition to meet.

The main consideration governing the change resulting in extending the main dam farther into the side hill was the reduction of the height of the embankment at the south end of the dam. The maximum height of the embankment above the level of the restored surface will now be about 50 ft. Another advantage gained is that the hardpan of the core-wall trench extends to the rock foundation for the full length of the trench, up to the juncture of the wall and the main dam.

* "Suggestions as to the Selection of Sources of Water Supply," by F. P. Stearns, M. Am. Soc. C. E. Report of Massachusetts State Board of Health, 1890.

Mr. Gowen.

The extensive bank of hardpan on the south side of the valley made it practicable to introduce the core-wall and embankment features into the design of the dam, and was, in fact, one great reason for seriously considering this location as feasible and advisable, under certain circumstances, in the beginning.

The core-wall is planned to stop at Elevation 200, as the water in the basin will not rise above this level except for short intervals of time, and the embankment, at this elevation, will be fully 130 ft. thick and well paved.

As to the possible upward pressure due to percolation under the foundations: Mr. Gould alludes to fears on this score as groundless. In this the writer fully agrees with him, but it must not be lost sight of that Mr. Deacon, in building the Vyrnwy Dam, established a series of collecting and discharging drains in the body of the dam, in anticipation of possible percolation tending to influence the structure's stability.

The query in regard to the cement used is pertinent, and a more definite statement is warranted. The Portland cement thus far used is American Portland, Giant Brand, while the term "American cement" alludes to light-burned cement. Of this, large quantities have been used—mostly of the Union and Bridge Brands—the former a Lehigh Valley and the latter a Rosendale cement.

Regarding Mr. Gould's reference to Fig. 2, Plate XLVI, and his criticism of the shape of the rubble stones used for the up-stream facing, it may be said that in the effort to build a face with as small an amount of joint surface (requiring spawling and pointing) as possible, the ordinary rules governing the proportions between the height and width of stones may have been ignored at times. This, it would seem, was warranted when the great thickness and monolithic character of the structure is considered.

The writer regrets that Fig. 1, Plate XLVI, should show a questionable streak near the middle of the unfinished end of the spillway. This streak was due to the wash of surface material from above, and he is glad to assure Mr. Gould that the workmanship at this point compares very favorably with that at any other point of the structure.

Mr. Rafter's account of the use of a rubber packer in connection with the diamond drill is interesting and valuable, and the writer fully agrees with him that through the water pressure tests opportunity is offered for very complete utilization of diamond drill borings.

The results shown in the quotations from the log of the pumping tests are remarkably well defined, and the back pressures obtained and flows traced, at a distance and at comparatively high elevations, are notable, if only as indicating the extent to which such tests can be used in the examination and tracing of particular seams, as well as masses of rock in general.

Mr. Gowen. In regard to Mr. O'Hanly's contention that there was no occasion for going below Elevation — 25 (the surface elevation of the limestone rock) for the dam foundation, and his reference to Professor Kemp's report to justify this position, the writer wishes to say that this report was quoted as being of interest under the circumstances, but that the paper is clear in showing that the investigation of the rock bottom revealed caves, fissures and seams of no inconsiderable size and extent, reaching in some cases to at least 40 ft. below the surface of the rock. In view of this, it is difficult for the writer to understand how at this time a course of action can be criticised which resulted in disclosing these defects which the report indicated as not likely to exist. That much of the foundation limestone was in a state of deterioration was evident, that some of it might have been in process of improvement is perhaps possible, but that the engineer should have accepted the disintegrated, seamy and fissured surface rock as a foundation, in view of a remote future possibility of change for the worse or better, seems indeed a novel idea. An engineer in so doing could certainly not be accused of timidity.

Lack of time prevents the writer from more than following the analysis by which Mr. O'Hanly has determined pressures at various points in the section of the dam, and he is willing to concede that a 1-in. cube of magnesian limestone will stand a crushing strain of many times the proportionate pressure on the dam foundations, but he fails to see any analogy between the crushing strength of a 1-in. cube of compact though not necessarily very hard stone, and the bearing strength of a mass of disintegrated seamy rock, through the mud-filled seams of which some percolation would certainly take place, with perhaps a possibility of washing out the earthy material.

Maximum pressures at the toe were calculated to amount to at least 15 tons per square foot, and the most ordinary prudence would seem to demand a reasonably compact and uniform bearing surface for a foundation.

Another point deduced by Mr. O'Hanly is that the weak point of the section is at Elevation 140, where he concludes that the line of pressure for a full basin falls slightly outside of the middle third. The writer is not disposed to admit this, as his records show that a recalculation of pressures, made some time since, when it was proposed to use the heavy "gabbro" stone for the hearting of the dam, indicates conclusively a well-defined margin of safety at the middle third at that elevation, as well as an excess of sectional area from an elevation above that point and extending to the toe. In this calculation the weight of masonry was taken at 166.67 lbs. per cubic foot. Mr. O'Hanly's assumption of high flow line at Elevation 210 is excessive, and may account for the difference in part of the results.

The triangular section derived from the Rankine formula has its

advocates, but, so far as the writer knows, it has generally been considered for high masonry dams in connection with a solid foundation, and as calculated to stand a possible full upward water pressure upon the base. In the section proposed, with the base at Elevation 40 and the thickness at this point, 240 ft., the area would approximate that of the section adopted for the New Croton Dam, and while there might be no question as to the ability of the ample section provided to withstand the pressure of the water, the efficiency of the proposed cut-off wall and the general effect resulting from the pressure of so heavy a mass of masonry upon a gravel foundation would be questionable. The uncertainty of results and the excessive section used would more than offset the saving of the excavation necessary to carry the other section to rock. Mr. Gowen.

The other triangular section proposed, with a width of base of 204 ft., at Elevation 68, while offering a reduced sectional area, would be open to the previous objections to a still greater degree. At this location it would show as a type of heavy masonry dam 142 ft. high, designed to resist a water pressure of about the same height, and resting on the ground surface.

The question as to whether dams so designed would be safe, raises the somewhat broader question as to whether any high masonry dam can be safely built upon other than a rock foundation. It seems evident that, to-day, practice answers "no."

The remarks of Mr. O'Hanly as to the desirability of adding to the paper details of the cost of the work, and men and plant employed, are fully appreciated by the writer, but such information to be of value must be spread in considerable detail, and would form an extensive paper by itself. The necessity of waiting until the work is finished in order that the information may be complete, and therefore more reliable, must also be apparent.